STRUCTURAL ENGINEERING

BOOK TWO

CONCRETE

BY

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STRUCTURAL ENGINEER

FOR

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Introduction.

This book is written to point the way to sound engineering in concrete by enunciating the principles thereof and by laying bare the falsity of much that passes for good engineering in this comparatively new branch of construction. The aim is to teach, not by example or model or system, but by laying stress on the principles that should govern in all design. One of the evils of following systems or models is that the adherent to a system is apt to use it where it is not appropriate and to hold to it, without alteration to suit the case, for fear that alteration—improvement—would be interpreted as confession of imperfection in the system. Principles are, or should be, of general application.

There are four general divisions to the book, as follows:

(1) The first part consists of information relative to the materials used in making concrete and reinforced concrete.

(2) The second part (pages 182 to 255 incl.), consists of articles by the author which were published in Engineering News (New York) in 1906, together with letters criticising the same, written by different engineers, with the author's replies to these. There are three of these articles. The one on Beams and Slabs was published in Engineering News, March 15, 1906; the one on Columns and Footings was published July 12, 1906; the one on Retaining Walls was published Oct. 18, 1906. The letters to the editor and the author's replies appeared in various issues in 1906.

(3) The third part (pages 258 to 413 incl.), consists of articles by the author which were published in Concrete Engineering (Cleveland, O.), in 1907, together with letters from engineers and the author's replies. There are eight of these articles, and they appeared serially, beginning with the issue of Concrete Engineering, Jan. 1, 1907, and being distributed through the larger number of the issues of that year. One article (Domes, etc.), will probably appear in the March, 1908, issue.

(4) The last part of the book consists of cuts showing piers, small arches, culverts, etc., as illustrating current practice. These are taken, with the consent of the publishers, from Engineering News, Engineering Record, Railroad Gazette, and Street Railway Journal, to whom the author hereby makes grateful acknowledgment.

There is some repetition in the book in the matter of the derivation of formulas for beams and columns, because of the fact that two of the articles in Concrete Engineering are on the same subject as articles in Engineering News. The repetition could not conveniently be avoided in this compilation.

Because some parts of the book are elementary in character does not signify that it is written for the tyro. There have been enough examples of flagrant blunders in public utterances and in design on the part of seasoned designers and eminent engineers to justify emphasis on ground principles that these utterances and designs violate.

No attempt has been made to carry through the book a uniform nomenclature. Values needed in any equation will be found close to it in the reading matter. The author has found attempted standard nomenclature extremely annoying. A practical engineer has not the time, when he wishes to make use of a formula, to read an entire book in order to make sure of the meaning of values in the formulas, and he is only wasting time when he must refer back to other chapters for their meaning. It is one of the greatest faults of books of reference, and text books that must often be used for reference, that formulas are set down with a view of their correctness, solely, the convenience of the user in applying them being ignored.

Attention is called to "Some Theses" in the section which follows. In the author's opinion so many of what ought to be well-known principles of mechanics are flagrantly violated in much current design, that the "investigations" and causes assigned when failures occur resemble the quibbling at a murder trial to determine whether the poison administered to a sick man, with "design," was the direct cause of death or the disease from which he suffered. Studied disregard of the greater menaces and magnifying of the lesser is not peculiar to investigations of concrete failures, however. Volumes have been published in the few months since the Quebec Bridge disaster on lattice bars and compression members. Some engineers hold that a left hand bottom chord *member, that* had been observed to be bowed,

was the initial point of failure. Others say that it was th corresponding right hand chord member, which was neve under suspicion, as this alone would account for the fa that the top chord of the bridge was thrown to the rigl a progressive amount (10 feet or so at the pier and 50 fe out where the traveler was located). Still others woul have both members give way at once, to account for tl failure of the bridge to lie down on the crippled side. Tl calculated unit stress in these chords at the time of failu is said to have been 12 000 lbs. A quiescent lead of th intensity would not, by anything known to engineers at tl present time, cause these members to fail. Fourte months before this disaster the author tried in vain (l a letter to an engineering journal, which they declined publish) to utter a public warning, pointing out the me ace of erecting this bridge with an immense traveler lacl ing sway bracing, or braced, if at all, with wire rope which would stretch excessive amounts. This traveler, the time of failure, held about 500 tons of steel more th 200 feet in the air, and no sway bracing, visible in any ph tographs, was used between the two bents. Beside tl roadway the vertical posts of the trusses had light latti in planes normal to the truss, which would offer but slig resistance to swaying, if the traveler should lean agair the top chord of the truss. In the course of tying up f the night (the wreck happened close to quitting time it would be a very natural sequence of events, that tl workmen should take a block from the left side of tl traveler and attach it to the truss at the right side, inten ing to make X-bracing thus, and that the tightening up the line should start the top-heavy traveler to moving t the right. The truss would be inadequate to stop it, an with just a little motion of the top chord the vertical pos of the truss would crush, doubling up on themselves. Tl same side force would bend in S-shape the two botto chord members in the anchor arm which were located the bottom ends and the junction of the first heavy dia onal and first heavy post of the anchor arm, causing the to fail at once, which they undoubtedly did, and alike, they also did. This hypothesis, which accounts for ever feature of the wreck so far made public, was ignored fc the untenable one, from all present engineering knowledg that two compression members, on opposite sides of structure, under about one half of their ultimate strengt or less, quietly gave way simultaneously. The predictio is here ventured that when this bridge is re-erected, tl traveler will be braced.

A Survey of the Field of Concrete Design and Construction, in which will be Found some Theses.

If the author were to write a history of reinforced concrete, he would be inclined to take his cue from the writers of school histories for juvenile instruction and make it a record of battles and mortalities. His own recollection of school histories is that they are not much more than a list of wars and their battles. It is hard to conceive what moral or intellectual benefit a child receives from loading his memory with names and dates of the battles and their casualities that exhibit the workings of the passions of men. In the matter of a new form of construction battles are inevitable. The passions of men exhibit themselves here as elsewhere; but the battles are with men who control older forms of construction, and little can be learned or gained by recording them. There are casualties from another cause, however, the record of which would be instructive. Reference is made to disasters that have resulted from errors in design or construction. A detailed record of these, with a correct statement of the fault, would be of great value. In many cases, however, if the truth were written, it would be a simple statement of dangerous conditions and of results that a study of these conditions would lead one to anticipate.

Accidents, of course, have played some part as the cause of failures, carelessness and ignorance have played a larger part. Among the causes assigned to failures in reinforced concrete the one most heard is "green concrete." This serves the purpose to the builder that "heart failure" serves to the physician. It is easier to blame the failure on some ignorant or dead workman who pulled out the props too soon, than to get down to the root of things and discover some vitally weak part of the design, that, even allowing months for the concrete to harden would have a factor of safety of but a little over one.

Another thing that is set down as the cause of mai failures is poor concrete. There is no doubt that failur have resulted both from the removal of forms before tl concrete has hardened and from criminal use of poor co crete. More often there can be found faults enough in d sign to cast suspicion back to one who ought to be posse sed of more intelligence than the workmen who are er ployed to carry out the design.

The importance of good materials and their prop manipulation cannot be too strongly emphasized, unless th emphasis obscures other conditions just as vital to t safety of the structure.

Many of the faults in design very often met with a pointed out elsewhere in this book; some of the most gla ing will be mentioned here at the risk of reiteration.

When a column is made of such light section that wood (which is both stronger in compression and immens ly tougher than concrete) it would appear too slender; ai when that column is made of plain concrete in which the are some longitudinal rods; and when it is true that su columns under a central load have been known to fail u der less compression than others of plain concrete devo of steel, the concrete in each being identical; failure is t most natural thing to look for, and that of an inclusi and disastrous kind. In Bulletin No. 10 of the Illinois E periment Station, page 14, column 8, a plain concrete c umn 9" x 9" by 12 ft. long stood an ultimate crushi strength of 2004 lbs. per sq. in. Column 2, identical size and having 4-⅝" rods embedded in the concrete, sto 1577 lbs. per sq. in. This is not an exceptional case. Oth series of tests have shown the same thing. It appears be the rule. Nevertheless, one reading the literature of i inforced concrete would be led to conclude that small lo gitudinal steel rods embedded in concrete columns a largely to the strength of the columns. Authority can found for including in the strength of a column the slender rods that would not stand alone and that the su *rounding concrete* supports in a very imperfect wa

Authority can also be found for including in a hooped column, apart from the compressive strength of the concrete, the strength of an imaginary column of steel, that is some function of a coil or set of hoops, which could have no strength whatever as a column except through the medium of the concrete, already given a value for its compressive strength. This assumed column of steel is purely imaginary. When phantom columns are relied upon to carry material loads, failure should not cause any comment. It is just as rational to make some of the links of a chain stronger than the others and then expect the strength of the chain to be greater, as it is to call the strength of a hooped column the sum of the compressive strength of an imaginary steel column, some function of the binding hoops or spiral, and of the concrete column. The latter is the only real column. All of the compression must go through it, and, while the compressive strength of the concrete is greater in flat discs between hoops that increased compressive strength is dependent upon the flatness of the discs and is not dependent upon the section of the reinforcement, as the formula would make it appear.

Much misapprehension exists as to the action of concrete in compression. In tension a bundle of independent fibres will carry a heavy stress. In compression a bundle of independent shafts of slender proportions would be of little use to carry a load. A material that is weak in tension cannot be very strong in compression in long members because of the tendency of the material to spread laterally and the need of tenacity to prevent this spread. Flat discs are very much stronger than cubes and cubes are stronger and more reliable than shafts a few diameters in length. Again shafts carrying loads not centrally placed are much weaker than the latter. Perfectly central application of the load on a column is something scarcely possible outside of a testing laboratory. Concrete, except in hooped columns, should not be called upon to take more than about 200 to 350 lbs. per sq. in. This applies to columns having only longitudinal rods. It also *applies to rein*forced concrete chimneys where

there is hooping; for the hooping in a chimney does not act like that in a column; it is merely to tie the concrete together, acting like the horizontal rods in a wall. It also applies to an arch under full thrust. In beams concrete is confined on all but the upper side; the extreme fibre stress in beams may therefore be much greater than in unconfined compression members. Unit stresses of 500 to 600 lbs. per sq. in. may be safely employed in beams.

When concrete fails in compression, spalls break off around the edge. It is plainly seen that in a flat disc these spalls are but a small fraction of the area in compression hence in such cases as a thin mortar joint heavy compressive stresses can be withstood. The same is true of the discs between two consecutive rings or coils in a hooped column. In cubes the spalls bear a larger relation to the total area, and in shafts the entire surface may spall off making the failure one in shear. The fallacy of using compressive unit values, found by testing cubes, to determine the strength of concrete shafts with or without reinforcement of longitudinal rods, is therefore apparent.

Shear in steel rods embedded in concrete at 10,000 lbs or 12,000 lbs. per sq. in. is one of the most blatant absurdities to be met with in literature on the subject of reinforced concrete and in building codes. It is totally unnecessary, even in an all steel structure, to specify a shearing unit on pins, for the reason that, if the bearing of the parts on the pin are properly proportioned, and the bending moment is not excessive, the section will of necessity be ample for the shear. Steel is thirty times as strong as concrete in bearing; hence there is something less than one-thirtieth of a reason for specifying any unit shear on the steel rods in concrete. If so-called shear rods are proportioned on the basis of 10,000 lbs. per sq. in., they are about as irrationally proportioned as they could be. Loose stirrups could not take hold of a horizontal rod to receive this shear, except through the medium of the concrete and, if the concrete takes the shear, there is no reason why

it should be imparted to the steel rods, since it must go back to the concrete again at the upper part of the beam.

T-beams, with their narrow stems and insufficient horizontal shearing area in a plane above the bottom rods, offer another example of irrational design. The apparent reinforcement supplied by vertical or diagonal shear rods is enlarged upon elsewhere.

As pointed out, wood is very much stronger than concrete. Wood will stand an ultimate load of 10,000 lbs. per sq. in. or more in tension and may safely take 800 to 1200 lbs. per sq. in. in compression. To make a reinforced concrete member capable of taking tensile stresses equal to what sound wood will take there should be 10 or 15% of steel reinforcement in a member of the same size. To reinforce a square member in bending, as with rods near each corner, so that it will be as strong as sound wood for transverse loading, as in a column taking swaying forces, it would require 5 or 10% of steel reinforcement. Reinforcements in such amounts are prohibitive; it is evident then that concrete members should be of larger dimensions than members in wood to perform the same office.

Reinforced concrete viaducts having bents composed of two inclined posts and a bottom strut, with the cross girder joining the tops of the posts, are not good construction. In wood such bents would be condemned for lack of diagonal bracing. Even in material of the toughness of steel such bents would not be attempted, unless diagonal bracing interfered with the headroom desired; in this case extra stiff columns would be used. More failures of structures have been caused by lack of proper bracing than from any other cause. The experience in the concrete engineering field should make all builders avoid, as dangerous, construction that depends for lateral stiffness upon long columns and lacks diagonal bracing. In the light of experience and intelligent interpretation of tests, as well as rational analysis, the building of such construction only invites disaster.

Reinforced *concrete does* not lend itself to efficient or

economic construction in open work resembling steel truss or bents. Girders or bents should be solid or nearly s Circular openings may be used rationally to save weigl

A common form of reinforced concrete column footi consists of a square flat block having rods lying near tl bottom, spaced uniformly, parallel to the sides. Some these rods are entirely outside of the column or sha carrying the load, that is, they do not lie under it and wou therefore serve to intensify the stress in rods that do l under the shaft. This is another example of irrational d sign.

There are a number of rudiments of popular but erron ous notions exhibited in much reinforced concrete desig One of these concerns sharp bends in rods. A hog cha or a king-post or queen-post truss are common forms supporting otherwise weak beams. A bent rod is throv under a beam, and at the bend a post is used to suppc the beam at one or two intermediate points. In reinforc concrete beams for test and in actual construction it is ve common to see rods similarly bent, and the appearan is that of the trussed beam just referred to. With this t popular eye would be satisfied, but when search is ma for something that corresponds to the post in the truss beam, nothing is found but a trifling amount of concre that occurs at the bend. To heighten the absurdity t so-called truss rod ends at or near the support with no e anchorage whatever. It is as though the truss rod in trussed wooden or steel beam were simply brought up a laid under the end of the beam without an end nut to ta the pull. A hook on the end of such a truss rod wou correspond to a hook in a rod embedded in concrete. Bc are weak and inefficient.

Sharp bends in heavy rods are very common; nothi could be more irrational.

It is a structural fault in a design when steel work so disposed that there must be some slip before the part c take its full stress. Such details are:—loose stirrups arou *rods, splices* in heavy rods made by lapping them a

binding them together, kinks or short bends in rods, etc.

Another concession to popular and erroneous ideas is in the use of arches where flat slab construction would be better. There used to be a bicycle manufactured that had a curve in one of the parts of the frame. Advertisements pointed out this weakening curve as a special element of strength, because it bore some relation to an arch. Segmental floor arches, lacking abutments and shallow where they need the greatest depth, and small arches of high rise, pressing horizontally against uncertain earth fill, are examples of the persistence of popular ideas.

When a beam may be severed from its supporting column or girder by the mere cracking of a surface of concrete and the pulling out of a short length of rod, and when this beam is an integral part of the only system of bracing in a structure, as in the case of a building where cross walls are not used to take up sway, it is reasonable to expect that wind or other lateral force, acting with the load on the beam will crack the beam and pull out the rod.

If reinforced concrete is to be used in high buildings to replace steel cage buildings, good features of steel frame buildings must not be overlooked. Specifications for steel work rightly restrict and forbid reliance upon tension on rivet heads to support vertical loads. But this very element is of utmost importance in holding together and bracing such steel structures as office buildings and mill buildings. Far too many reinforced concrete structures, lacking a unifying element tying the various parts together, have ingloriously failed, and far too many laborers and foremen have been made to take blame that should have been attributed to ignorance of designers. It is of utmost importance that the parts of a reinforced concrete building be tied together. This should be done by use of continuous or spliced rods. The best way to accomplish this is to use round rods and to splice them with sleeve-nuts.

Nowhere in engineering is there more evidence of a disposition to cover up ignorance with elaborate cloaks of arithmetic fabric than in reinforced concrete design. This

is exhibited in complex formulas for the design of beams, in the flat plate theory, in arch calculations, in column formulas, in calculations for temperature stresses and generally where concrete and steel design is treated. Some of these have already been referred to. The most discreditable feature about it all is that structures that have fallen down have been shown by some of these applications of arithmetic to be quite safe.

Another thing in evidence is erroneous formulas and misapplied methods of calculation.

An illustration of what elaborate theory accomplishes in beam design is seen from the following example, taken at random. In Bulletin No. 14, Engineering Experiment Station, University of Illinois, page 11, test beam 57 shows a calculated unit stress in the steel of 39,800 lbs. This is worked out by a formula with many terms and all of the commonly made assumptions that bring in the relative moduli of elasticity of steel and concrete. The test load is 11,730 lbs. at third points on a 12-ft. span. Adding to the moment that this would give the dead load moment due to the weight of the 8″ x 11″ beam there is found to be a total moment of 300,900 in.-lb. Taking the neutral axis of this beam at the middle of the depth of the concrete, as the rods are 1 in. above the bottom, the effective depth is 8⅙ inches. The stress on the steel is 36,800 lbs., or on three ⅝-in. rods, 40,000 per sq. in. This is one-half of one per cent. more than the unit stress found by the elaborate formula. The results are thus practically identical in this case at least.

The flat plate theory, as worked out for such homogeneous material as steel plates, is given in all its elaborateness, as applicable to reinforced concrete slabs supported on all sides, by a number of writers. In the theory of the flat plate, as usually treated a factor enters that does not find a parallel in the ordinary theory of flexure. At the center of the plate the extreme fibre stress at top and bottom is of the same intensity in all directions, in the two respective planes. Because of this fact the upper layer is capable of

resisting a greater intensity of stress than it would take with the stress in one direction only and the material not confined laterally. The fibres in compression in a flat plate are confined on all sides, including, to some extent, the side on which the pressure is applied. In the case of the fibres in tension there is not this assistance from conjugate stress. It is not clear, therefore, why Poisson's ratio should be applied to flat plates, since the tension on the free side would be the critical stress. The term representing fibre stress in the flat plate theory as commonly given is not the true fibre stress but a fictitious unit stress resulting from the application of Poisson's ratio. The actual fibre stress is almost cut in half by this application. Advocates of the flat plate formula do not seem to have considered the question of converting this fictitious extreme fibre-stress in all directions into actual stress in reinforcing steel rods in two or more directions. They further do not make it clear why all of the complexity and uncertainty of the flat plate theory for steel plates should be resorted to to solve flat slabs in reinforced concrete, a material not at all contemplated in such formulas as Grashofs.

Some prominence has been given recently to a solution of the flat slab supported on four sides, which starts with the assertion that a flat square slab supported on all four sides fractures along a diagonal. Assuming that the bending moment is greatest along the diagonal of a rectangular slab a formula is derived for the magnitude of this bending moment, which is naturally a mathematical certainty, just as the bending moment along the diameter of a circular plate is a mathematical certainty. But the variation of intensity of this moment along the diagonal is entirely ignored, or rather the intensity is taken to be constant. There is just as much error in taking this moment as uniform along the diagonal of a rectangle as there is in taking the intensity of the moment along the diameter of a flat circular plate as uniform. It may be true that a flat slab, either square or rectangular, supported on four sides, will break on a diagonal line. This is proof of one thing, name-

ly, that the greatest intensity of bending moment is at th center of the slab; for, as both diagonals are identically con ditioned, either may be the one to fail, and their only com mon point is the center of the slab. It is not conceivabl that the same intensity of bending moment extends clear t the corner of the slab. A crack once started in a diagona direction would naturally extend into the corner and eve over the support, merely on account of the brittleness o the material. This would be no evidence of constant in tensity of bending moment along the fracture; much les force is needed to continue a crack once started than t start the same.

The author brought out a formula for square slabs sup ported on four sides and reinforced in two directions, i Concrete Engineering, Feb. 1, 1907. This formula is appl cable to the materials considered and rational in its der vation. It has the sanction of Professors Maurer and Tur neaure, as it appears in their book recently published.

In the design of arches the elastic theory has been em phasized and very forcibly recommended. This is a theor for the investigation of an arch already proportioned. could not be used to find the proportions needed. One wa recommended to arrive at a trial proportion for an arc is to take the dimensions of existing arches and judge therefrom. An inspection of a table of the proportions existing arches reveals the fact that they are hopelessly variance with each other. The elastic theory requires th the moment of inertia of the arch at all sections be know The moment of inertia of a combination of concrete an steel can be assumed, or arrived at by assumptions, but n definitely known. The elastic theory, as usually employe assumes fixed ends to the arch. This is an extravaga assumption and one not warranted by the conditions. small models it would be possible to make mass enough the abutments to effect fixed ends in the arch span. large arches it requires massive abutments to resist ove turning from the simple thrust alone. If to this, mass mu be added to make the ends of the arch fixed, an unwarran

ed waste of material results. The concrete needed to add a few inches to the depth of an arch ring and thus increase by a large percentage its stiffness would be insignificant in results if added to the abutments. The thrust, if the arch be considered hinged at the ends, is practically the same as if it be considered fixed ended, so that the amount of mass in the abutments due to assumption of fixed endedness is simply extra material that could be more economically employed in the arch ring.

The elastic theory has application to steel arches for two reasons. (1) The moment of inertia of a steel arch can be definitely calculated. (2) The unstressed arch fits the abutments upon which it rests, being very accurately built therefor. This theory is inapplicable to stone or concrete arches for the obverse of these reasons. The shrinkage of concrete in setting precludes any possibility of determining exactly what would correspond to the moment of inertia of a reinforced concrete section, that is, a coefficient from which the deflection could be calculated. This same shrinkage makes it impossible to build a concrete arch of any kind that will be free from shrinkage stresses, and an arch in which there are unknown initial stresses should not be made the subject of exact calculation based on the absence of initial stress. Inaccuracy in stone arches has the same result as shrinkage in concrete arches in that the arch is subject to some initial stress. In addition to this stone arches cannot take any tension.

In the inelastic theory the presumption of accuracy is not made, and there is not the false security which elaborate calculations tend to give. It is more in keeping with the materials of stone and concrete arches. A system of blocks fitted to each other along the lines of an arch will carry certain fixed loads without tending to open any of the joints, when these blocks follow the line of what is called the equilibrium polygon. When other loads are brought on the arch, as the live load, the equilibrium polygon undergoes certain changes which would tend to open joints at sections whose location is moderately well known. If the

arch blocks be conceived to be cemented together into one mass and to be tied together by steel rods, the rods will take the tension. Reasonable assumptions to arrive at the probable intensity of the tension are all that the materials and methods of manufacture in reinforced concrete arches justify.

The literature on stone and concrete arches is unsatisfactory being in large part excessively theoretical and unpractical from many standpoints. Semi-elliptical and basket-handle arches are commonly held out as examples. Such curves are unsuitable for ordinary arches, because they presuppose heavy horizontal pressure of the fill or demand the same to prevent large moments at the sharpening of the curve. Only sand or semi-liquid mud can exert such pressure (except as a wedging force, as back of a retaining wall) and neither of these materials is desirable as a fill. If the arch is the roof of a tunnel under a stream, the semi-ellipse or semi-circle are consistent curves. There is a general lack of appreciation on the part of writers and builders, of the fact that horizontal pressure in earth, while it is a possibility, is not necessarily exhibited. There are many proofs of this, but they pass unnoticed. Ordinary earth could be supported on a system of vertical props capped with horizontal lagging pieces in approximate arch shape. Unless earth will loosen in large chunks, there is scarcely anything to fear from horizontal pressure. This action is not possible in the fill over an arch. Horizontal pressure in such case is scarcely a possibility. Of course horizontal motion against earth would meet with more or less resistance, but this is passive and not active.

Another phase of the impracticable nature of data on arches is seen in the blanket formulas for depth at crown. These often have no relation to the load to be carried, whether heavy or light traffic. Some of them give the depth of ring in terms of the radius of the curve of the intrados, and usually the rise is taken from the point where the intrados begins to curve away from the face of the abutment. These details of outline have no significance as

determining the stresses but are only architectural features. The curve of the intrados may have only a remote relation to the real curve of the arch, which is the curve of the central line of the arch ring.

In another respect literature on arches is unsatisfactory. This concerns live load stresses. In designing girders and trusses in steel, the live load can be placed at the exact point where it will give the maximum stress in any member. To do this, using the elastic theory in arches, would introduce frightful arithmetic complications. No systematic attempt seems to have been made heretofore to determine the position of concentrated loads to give the maximum effect on stone and concrete arches.

Further it has not been recognized in treatment of arches that there is a rise for any given span and loading and fill which fits the conditions, that is, a point where, if the rise is diminished, the thickness of arch ring (for thrust) will be unnecessarily great to take care of eccentric loading; and if the rise be increased the thickness needed (for thrust) is not enough to take care of unbalanced loading. This is the natural economic rise for any span.

Calculated stresses in concrete arches due to temperature changes are void of meaning. If a concrete arch could be made apart from the site and then placed bodily in its intended position, and if it fit exactly that position, calculated temperature stresses might have a meaning. It is impossible to place concrete in such way that its normal unstressed shape fits exactly on the supports. It is equally impossible to determine the intensity of stresses due to shrinkage. If then the original condition of stress cannot be known, it is idle to make elaborate calculations as to the effect of expansion and contraction due to change in temperature. In steel work arches can be made to fit the supports, and temperature stress calculations have a meaning. A liberal factor of safety is eminently better to cover ignorance than the most ingenious mathematical fabric ever devised.

When a dam fails, elaborate theories are put forth to ac-

count for its overturning, when calculations show that tl horizontal pressure of the water is not sufficient to tip over. Suction on the down-stream face is one of these bı bears. There has been found to be a pressure somewh below the atmosphere under a falling sheet of water whe the stream contracts, and this is seized on to account fı the great force that would be necessary to overturn a ma of masonry. Or in the matter of strength shearing forc in vertical or horizontal planes are blamed for the failuı In technical literature on the subject, so far as the authoı somewhat indifferent search has shown, there is no mentiı of the uplifting tendency of water that percolates under tl dam, supplying nearly or quite half enough upward force lift the masonry of the dam. This force, in a gravity da designed to resist both the uplift and the lateral force the water, is 42% of the total overturning force or 72% the horizontal force. A system of design that ignores forc of such magnitude as these and magnifies trifles is not safe system even for a temporary structure, to say nothiı of the danger in a permanent structure upon which haı so much life and property.

A scheme for reinforced concrete retaining walls, consi: ing of a curtain wall and a bottom slab joined at interva by ribs or counterforts, has been much used in recent yea: Two errors in design characterize many of the desig that have been described in engineering periodicals and books. One of these concerns reinforcement of the bottc slab. Complete reinforcement, uniformly distributed f the full width of slab, is used near both top and bottc surfaces. No analysis of the forces can show need of su wasteful use of steel. In the matter of reinforcing the ı or counter-fort the apparent and expressed method is treat it as a beam with increased tensile stress from en toward middle of the inclined edge. This is far from cı rect. The stress in the reinforcing rods is imparted at thı ends and failure to use positive end anchorages is a struı tural blunder.

In formulas for reinforced concrete chimneys much u

cessary complication is sometimes introduced by making the neutral axis out of the center of the section. There is no good reason, based on known facts, for assuming the neutral axis anywhere but in the center of the section.

There is much misapprehension in the matter of interpretation of tests. Some published tests are not worthy of the name of tests. They are made for the purpose of "proving" the miraculous strength of some system, and the load is improperly placed or is placed on only a portion of the floor. Tests of shear in concrete are referred to at length elsewhere. Some recent tests were reported that purported to give the adhesion of steel to concrete. The entire resisting moment of reinforced concrete beams (with short lever arm) was attributed (so far as tension was concerned) to the reinforcing steel rods, no allowance being made for the tensile strength of the concrete, a real though uncertain quantity. By using smaller and smaller rods such tests could be made to "prove" the adhesion of steel to concrete any desired amount; for even with no rods whatever, the plain concrete would take a certain bending moment. In one very important respect the lesson learned from tests is a perverted one. Reference is made to the test of practical use of a structure. There is no more misleading notion than the one that because a structure stands and performs its office it is therefore safe. Structures have collapsed after standing for decades, and this under no unusual conditions. Given a structure that has attained almost its full strength, and suppose that this structure fails when the forms are removed. What would be the factor of safety in such a structure (or in another similarly built). If it reached a strength 10 or 20% greater and was capable of standing up when the forms were removed?

As to the unit tension allowed on steel, this is also elaborated elsewhere. Units above 10,000 to 13,000 lbs. per sq. in. are too high, though they are very generally recommended. At these units the concrete will generally retain its integrity. Not that steel can always be stressed to this amount without cracking the concrete, but, if there is

enough concrete surrounding the steel, the tensile strength of the concrete will aid the steel to the extent, for safe loads, that the elongation will not crack the concrete. A design that contemplates cracks in the concrete under safe loads cannot be classed as good engineering. The cause that will produce a crack at one point may produce others at other points and thus break up and wear out the structure.

In the matter of the consistency of concrete there are still some users that insist on "moist earth" mixtures for all purposes, even reinforced concrete and parts that ought to be impermeable. Also it is very generally recommended that materials be heated before use in cold weather. A very significant letter appears in Eng. News, Jan. 30, 1908 from Mr. E. A. Mollan. He describes some concrete that was made with sand and stone that had been dried out by the heat of the sun. This concrete had no cohesion and was worthless. When, subsequently, the same materials were wetted down concrete made therewith (using the same cement as before) was of good quality. Builders who make use of stoves to heat and dry the sand and stone for concrete are commended to a study of this case.

Cement.

There are two kinds of cement in common use, namely, sendale or natural cement and Portland cement. To s may be added, also, slag cement or puzzolan cement.

Rosendale cement is made by burning a limestone conning the carbonates of lime and magnesia and clay at out the temperature of the lime-kiln or 1000 to 2000 grees F. It is ground to a powder between mill stones ter burning. The color is usually brown. Rosendale ment is used to some extent in foundations for street vements and in some massive concrete work. It should t be used in reinforced concrete work or in work where rength is to be an important characteristic.

Portland cement is made by mixing clay and limestone or other argillaceous and calcareous substances) in the oper proportions, and burning the mixture at a high eat (above 2000° F.), and grinding the clinker to a power. The color is usually gray or greenish gray. There e four different kinds of Portland cement manufactured the United States as distinguished by the materials om which the cement is manufactured. These cominations are: 1. Argillaceous limestone, or cement rock, nd limestone. 2. Marl and clay. 3. Hard limestone and nale or clay. 4. Slag and limestone.

There is a kind of cement called slag cement, or puzzoin cement. This is made by first granulating the slag y chilling the molten slag in water, then drying and mixig with slaked lime, then grinding. This is an inferior cement and is only good in locations where it will be contantly wet. The color of slag cement is light lilac. Slag ement should not be confused with true Portland cement nade from slag and limestone. The latter is made by urning to a clinker a mixture of crushed limestone and hilled blast furnace slag and then grinding this clinker s the clinker of other Portland cement is ground. A lescription of the manufacture of this kind of cement is iven in Engineering News, Sept. 27, 1900.

These cements are all what are called hydraulic cements, that is, they will set or become hard under water. They are unlike common lime in this respect, as they do not require the presence of air to acquire their cementing quality. The presence of water is necessary to the hardening of cement, and cement will harden both in fresh or salt water. Unlike lime, also, the clinker from which cement is manufactured is inert. The unground lumps are like cinders, and they are therefore useless in this state. It is only when the clinker is ground very fine that it is fit for use as a cement. This is the reason why fine grinding is essential in all hydraulic cement.

An average analysis of good Portland cement is about as follows: Lime, 64%; Silica, 21%; Alumina, 8½%; Magnesia, 2½%; Iron Oxide, 2½%; Other ingredients, 1½%. A variation one way or the other of 2% or so in the amount of lime and silica does not make much difference in the cement. The magnesia should not exceed about 2 or 3%. There is not so much regularity in the composition of Rosendale or in slag cement as there is in Portland cement. There is generally much less lime and more silica and magnesia in Rosendale than in Portland.

Portland cement is superior to Rosendale for every purpose, for a given volume of cement, though the latter for many uses meets the requirements of the case. Natural or Rosendale cement sets quicker than Portland cement, though Portland cement soon overtakes and passes the natural cement. The quick-setting property of natural cement may be turned to advantage in work that is to receive its load soon after placing. By using a larger quantity of natural cement than would be needed in Portland cement the necessary strength may be attained in a short period.

Natural and Puzzolan cements will not stand extreme changes in temperature as well as Portland cement.

When sand and cement are mixed and ground together, the mixture can be ground finer than it is possible, with

the same means, to grind the cement alone. It can in fact be ground so fine that nearly all of it will pass through a sieve having 200 meshes to the inch. On account of the extreme fine grinding, and possibly also on account of the activity of the sand in this finely divided state, the cement is stronger than the same amount of ordinary cement would be. If the installation of a grinding mill is practicable on a large piece of work, where the cost of cement is high, there may be a saving in thus grinding a mixture of cement and sand in order to reduce the amount of cement required. The grinding of a mixture of cement and sand is patented.

The weight of Portland cement is 85 to 100 lbs. per cu. ft., depending upon whether it is loose or compact. A barrel of cement is considered, by some specifications, as equal to four cubic feet. This, at 375 lbs. per standard barrel, is about 94 lbs. per cu. ft.

The tests commonly made on cement are given elsewhere in this book. A few notes on the method of making these tests and the importance of the tests would not be out of place here. Some additional tests will also be mentioned.

To determine whether or not a cement is hydraulic mold a brick 1″ x 1⅛″ x 8″, and after the initial set place it under water upon supports near the ends, having the one-inch dimension vertical. If the cement is hydraulic the brick will retain its shape, if not, it will give way between the supports.

A simple test for soundness of cement, or freedom from tendency to shrink or expand during setting, is to take a cylindrical lamp chimney and fill it for a certain distance with well compacted cement paste, marking the end of the flat surface. If the cement shrinks, it will show by the mark, and if it swells, it will break the chimney.

The swelling of cement during setting is usually due to the slaking of free lime. When the cement is used fresh from the mill it is apt to have some free lime in it. Seasoning for several weeks after grinding, if the cement

is finely ground, will air slake this lime and render the cement more sound. On the other hand too long exposure to the air will destroy the activity of the cement and may render it useless. Moist air is especially destructive on the cement. It should therefore be stored in a dry place. Air generally affects cement by causing it to cake in the sacks, and the hard lumps that cannot be easily broken with the shovel will be useless as cement. Good cement may be somewhat lumpy, but the lumps should be such that they can be easily broken with the fingers.

The test for fineness of cement is important, because the finer a cement is ground the stronger that cement will be. Not only is the activity of the cement increased by fine grinding, but the fine particles are necessary to fill small voids in the sand and thus render the mortar or concrete dense.

The test for specific gravity is made to determine whether or not the cement is properly burned and also to detect adulterants. If Portland cement is under-burned, its specific gravity will be low, and if it is over-burned, its specific gravity will be high. Specific gravity tests are apt to be misleading, because of the fact that the cement absorbs some CO_2 and water from the air and thus its specific gravity is reduced. Cements having different degrees of calcination, if tested when fresh-burned, or if heated red hot before testing, to drive off absorbed CO_2 and water, appear, according to Mr. David B. Butler (Eng. Record, Vol. 55, p. 176) to have very close to the same specific gravity. Mr. Butler's tests, however, do not discredit the value of the specific gravity determination to detect adulterants. Natural cement or slag used as adulterants will lower the specific gravity.

The conclusions in a paper by Prof. R. K. Meade and Mr S. C. Hawk, read before the American Society for Testing Materials in June, 1907, drawn from a series of tests made by them, are as follows:

"(1) That the specific gravity test is of no value whatever in detecting underburning, as underburned cement

will show a specific gravity much higher than that set by the standard specifications. Underburned cement is readily and promptly detected by the soundness test and no others are needed for this purpose.

"(2) The value of the specific gravity test as an indication of adulteration is much exaggerated. While a large admixture of any light adulterant with the cement would be shown there is at the same time much slag cement and also Rosendale cement which could be mixed with cement in large quantities without lowering the specific gravity below the limit of our standard specifications.

"(3) That low specific gravity is usually caused by seasoning of the cement or the clinker, either of which improves the product.

"(4) That the proposition to ignite the cement sample which falls below specifications and determine the specific gravity upon the ignited portion is of no value because adulterated cements also have their specific gravity very much raised by such ignition.

"(5) That the requirements for specific gravity should be omitted from the standard specifications or at least that the clause which infers that low specific gravity is caused by underburning and adulteration should be omitted and that in its place there should be inserted one stating that low specific gravity may, but does not necessarily imply adulteration as it is in most cases due to seasoning of the cement or storage of the clinker before grinding, both of which are beneficial to the product."

Specific gravity of cement is usually determined by immersion in benzine or turpentine.

The addition of a small quantity of unburned granulated slag, before grinding, to cement made from slag is not considered an adulteration, as it neutralizes the effect of any free lime that may be in the cement.

Cement that shows excessively high tensile strength in short time tests is liable to be adulterated with sulphates, which hasten the settling but render the cement weak after a long time.

In making pats or bricquetts for test the paste should be mixed thoroughly for five minutes, rubbing the mixture under pressure. Regularity should be observed in placing the mortar in the molds. Pressure should be used, and as near the same pressure as possible for different tests. A difference in the amount of tamping or pressure may make a very great difference in the strength of tensile tests.

Soundness of cement is of more importance in rich mixtures than in lean mixtures. Cement that will stand the 28-day test for soundness, but fail under the accelerated test by boiling or steaming, may be satisfactory in ordinary concrete. Cement that will stand both the accelerated and the 28-day test is to be preferred. Cement that shows up well in the accelerated test is not necessarily a good cement. The presence of sulphates seems to counteract the expansive effect of free lime. Gypsum or sulphate of lime is generally added by manufacturers of cement to retard the set. Sulphate of lime to the extent of more than two per cent. by weight is not allowed in any cement by the U. S. Army engineers; not more than one per cent. is allowed in cement to be used in sea water.

Cement that shows up well in the tensile test after seven days and then shows little or no gain at 28 days is to be looked upon with suspicion.

The tests given in cement specifications are to be made in a laboratory equipped for the purpose. Simple tests may be made on the work, that will in some cases be sufficient to determine the general character of the cement. Pats and balls of cement or mortar can be made use of to gage the setting qualities and soundness. By pressure of the thumbnail the setting quality may be estimated and by dropping the hardened specimens a rough idea may be had of the strength.

Puzzolan or slag cement may be detected by taking a pat and boiling it for several hours and then breaking it. The fresh fracture will be bluish green.

The following is quoted from "Professional Papers of

e Corps of Engineers, U. S. Army. No. 28." "Puzzolan ement never becomes extremely hard like Portland, but uzzolan mortars and concretes are tougher or less brittle than Portland. The cement is well adapted for use in sea water, and generally in all positions where constantly exposed to moisture, such as in foundations of buildings, sewers and drains, and underground works generally, and in the interior of heavy masses of masonry or concrete. It is unfit for use when subjected to mechanical wear, attrition or blows. It should never be used where it may be exposed for long periods to dry air, even after it has well set. It will turn white and disintegrate, due to the oxidation of its sulphides at the surface under such exposure. Sulphuretted hydrogen, which is often evolved upon decomposition of the sulphides in Puzzolan cement, is injurious to iron and steel. Such metals, if used in connection with Puzzolan cement, should be protected, or an allowance be made for deterioration by an increase of section."

Cement is considered by most authorities to be a substance which attains its hardiness by a process of crystallization, taking up the water used in mixing to form a hydrate of a crystalline structure. The author believes with Dr. William Michaelis of Germany that cement does not crystallize in hardening, but that it is a colloid. The author believes that the nature of cement is more nearly represented by ordinary glue than by any crystalline substance, and that what crystallization takes place is accidental and not a necessary accompaniment of the hardening process. Cement clinker is inert. It does not take like quicklime: it must be first ground to a powder to have any cementing quality. It is only the [illegible] powder of this so-called cement that is readily [illegible]. The rest is inert, like the clinker. Pure neat cement, [illegible], would not get very hard and would [illegible] Laitance, or the slime that rises to the surface [illegible] ment mixture in an excess of water, is the nearest [illegible] to pure cement that is usually [illegible]

neat cement is a mixture of small particles of inert clir er and a dust that is fine enough for water to act up and turn into a colloid or gellatinous substance, whi when lodged in between small grains of an inert hard su stance, holds these grains together in an artificial sto The inert hard substance may be grains of sand or t coarser grains of ground cement clinker, too coarse have cementing properties.

The foregoing statements are radically different fro the commonly accepted belief regarding cement. Th therefore call for some facts to substantiate them. attempt will be made to discuss the chemistry of ceme except to cite some admitted facts that bear in a gene way on its chemical action.

It is well known that fine grinding improves the streng of cement. It is also known that there is a point beyo which fine grinding diminishes the strength of ceme when made into neat briquettes and pulled. It is a known that this same finely, ground cement, that is wea er in neat tests than coarser cement, is stronger in ment and sand tests than the other.

Another fact well known is that when a broken pie of china is to be mended by glue, or when two pieces wood are to be glued together, the parts must be press firmly together and as much of the glue squeezed out possible; and the strength of the mended part is grea than that of the glue itself by which it was mended.

Lime mortar, even before it has had time to be ac upon by the carbon dioxide, is stronger in tension th the lime paste.

Melted sulphur is used sometimes as a cement, usua however, alone. In Engineering News, Vol. 51, p. 2 Mr. Alexander Potter describes some tests on melted s phur as a cement for sewer pipe, mixed with sand. finds it to be an excellent material for that purpose. his tests he found plain sulphur to resist tension with ultimate strength of about 100 lbs. per sq. in. Sulphur a *sand mixed* hot, 1 of sulphur to 1 of sand stood 650 l

sq. in.; 5 of sulphur to 7 of sand stood 670 lbs. per sq. . 2 of sulphur to 1 of sand stood 400 lbs. per sq. in. . Potter found that fine sand such as quicksand gives ter results than coarser sand. These strengths were wn as soon as the sulphur was cold. This is analogous the action of Portland cement. When very finely und, it has an overdose of cement and not sufficient rt substance to be cemented together. When sand added till a balanced mixture is formed, the mortar strong. Sometimes sand and cement tests pull up onger than neat cement. Cement reground with sand made much finer and will take in more sand to form a lanced mixture. The mixture may be unbalanced by ving too much cement, as in the sulphur tests, where ore sand added makes the specimen stronger; or it may unbalanced by having too much inert substance to have e interstices filled by the cement, as in lean mixtures of nd and cement.

That Portland cement is a mixture of a cementing subance (the finest powder) and an inert substance may be own by the following experiment. Take a little cement nd mix it in about ten times its volume of hot water. et this boil for several hours, adding more water as reuired. At the end of this time stir up the mixture and our it rapidly into a tumbler. Almost immediately there ill be a settlement of part of the cement, which is of a ark color. Upon this there will settle a light colored art of the cement, leaving the water comparatively clear. distinct line will separate the dark colored and the light olored parts. The upper layer will be slimy at first then ellatinous, and will finally (in a week or two) attain a aty hardness. Upon drying it becomes like chalk and as little cohesion. The lower layer, which in an experiient by the author was about equal in volume to the upper layer, is like sand and has little cohesion. It can be rumbled in the fingers. The upper layer is the slime or itance or true cement that is liberated in an excess of ater, when concrete is mixed. The fineness of the grinding

will no doubt gage the relative amounts of inert and active substance in the cement. In a repetition of this experiment the cement was allowed to settle in a glass, and the water was poured off. The separated cement was then mixed together into a mortar and allowed to harden. It hardened into a cohesive and hard lump.

By the use of cold water and long continued agitation a partial separation can be effected, but enough of the fine dust seems to adhere to the inert particles to cement them together. The light and dark colors will be present but they will not be so clearly defined.

Some tests presented in a paper by Messrs. Henry S. Spackmand and Robert W. Lesley, read before the Association of American Portland Cement Manufacturers at their annual convention in 1907 (See Eng. Record, Dec. 21, 1907, p. 691,) prove very conclusively that a large part of commercial Portland cement is inert on account of not being fine enough to have any activity. Briquettes of neat cement were made and at periods of from 7 days up were tested. At 28 days they stood a tensile test of 752 lbs. per sq. in. These broken briquettes were then dried and reground. This material was used as cement and made into briquettes, which stood, at 28 days, 253 lbs. per sq. in. These broken briquettes were dried, reground, and used again as cement. The briquettes stood 163 lbs. per sq. in. at 28 days. Tests were made at other periods, and mortar tests were made with sand. The above illustrates what the tests demonstrate. In the words of the experimenters—"These experiments clearly show that even after cement has been twice gaged with water and allowed to harden under water, all the cementing and hydraulic qualities are not destroyed." It is clear that the regrinding of the briquettes broke up particles that were not fine enough to be active at the first and second gagings.

The paper above referred to describes other experiments that further strengthen the conclusion that the finest ground Portland cement is composed largely of inert clinker, too coarse to be active. Quoting the paper again.—

ɔrtland cement which had passed the 200-mesh sieve s further separated by elutriation into the following ts: (A) Material that settled out in 30 seconds;) Material that remained in suspension for 30 seconds, : settled out in one minute; (C) Material that re-ined in suspension for more than one minute. The ce-nt thus divided into three portions according to size s treated with water in tightly stoppered tubes. "A" s only slightly acted upon by water, even after two rs contact with it. "B" was only acted upon by water er three or four months, and only a portion became lly hydrated. "C" was acted upon almost immediately, elling up and forming a very voluminous jelly." [Elu-ation is shaking up in dry kerosene and allowing to set-. The amount of this cement that settled in 30 seconds not stated. In another sample 45.18 per cent. settled in is time.]

When a substance crystallizes, the crystals assume a efinite volume and have not the property of swelling to larger volume or of occupying less space. Further, these ystals have the property of exerting pressure to reach at given volume, as exhibited in crystals that form in e pores of soft stone swelling with such force as to crack e stone. Sound cement will not swell with force as rystallizing substances do. It is true that cement that ardens under water will swell slightly, but the same ce-ent hardening in the air will shrink, proving the lack f definite volume in the hardened cement. On the other and cement that is not confined by the inert particles ressing against it can be made to swell to fill a space uch larger than its original volume. The minute grains f cement, if they are finely broken up, have the property f swelling to many times their size by the absorption of ater and hardening in water or in air. If abundance of ater be present, the grains will swell up to a larger size nd make a more dense and a stronger mortar; and, if he water be present for a sufficient time, the cement ill unite with some of it and harden in this larger vol-

ume. Upon being taken out of the water after being thoroughly hardened the cement does not then contract in drying out, but the surplus water dries out of the minute pores in some such way as a cork that has been soaked in hot water dries out, though the water would not pass through the pores of the cork in a liquid state.

The amount of water permanently retained by the cement in an ordinary concrete mixture or in mortars of about 1:3 is about 16 to 18 per cent. of the weight of the cement. In neat cement only about 8 to 12 per cent. is retained. This is further evidence of the fact that the hardening of cement is not a crystallizing process. Crystals have a definite amount of water in their composition. It is evidence also that the pure cement will swell and absorb water in proportion to the space to be filled and the amount of water present, as the voids in mortar are much greater than in neat cement; also the amount of water present, even in mealy concrete, is greater than needed in neat cement mortar. These facts would further seem to vitiate the contention of those who say that only the bare amount of water for the needs of the cement should be used in the mixing, since cement under varying conditions will combine with different amounts of water.

When cement is mixed with a meagre amount of water, the grains do not have the conditions present that promote free swelling to fill the voids. The result is that the mortar or concrete is porous, that is, full of large pores that allow the passage of water. It is an erroneous belief that the impermeability of concrete is promoted by using a meagre amount of water in mixing and tamping this concrete.

It is well known that concrete can be made quite good that is deposited in water and it is well known that specimens that are kept in water after setting will, upon hardening, be very much better and stronger than like specimens hardened in air. It is not clear why authorities, in the face of these facts, warn against the use of plenty of water in the mixing.

Dr. William Michaelis, in a paper read at the annual meeting of the Association of German Portland Cement Manufacturers at Berlin in Feb. 1907 (See Cement and Engineering News, Aug. 1907) describes a piece of artificial stone made of Portland cement and 10 or 20 per cent. of asbestos fiber. The mixture was stirred a long time, as paper pulp is stirred, and then pressed to expel the surplus water. The volume of the stone was many times that of the volume of the cement used in the mixture, showing that the cement had swollen by the absorption of water to attain this volume. Dr. Michaelis says of this stone that it is rightly named eternite, for it is practically everlasting. The stone resembles slate. There is a company that makes this kind of artificial stone for shingles. According to those who advocate a mealy mixture for concrete this stone would be expected to be porous and totally unfit for shingles on account of the large excess of water used in making it. The presenec of the asbestos fiber and the long continued agitation hold the grains of cement apart and allow them to swell by the absorption of water.

Pure cement when placed in water will swell to many times its volume measured as a dry powder, if not confined. In commercial neat cement, however, the particles of inert clinker act to confine the pure cement and prevent the swelling. If there is plenty of water in the mixture, and the cement is well mixed through the mass, the cement will swell to fill the voids and will bind together the inert particles making a stronger and better concrete than can be made by a mealy mixture, even with pressure applied in the manufacture. If the swollen cement dries out before it has hardened, the concrete will shrink. This is the reason why cement or concrete hardened in water will swell, and, if hardened in the air, it will shrink. This is the cause of shrinkage cracks in work that has not been kept moist during hardening.

Tests for soundness or constancy of volume in neat cement may be *misleading*, because of the fact that the

best cement, that is, the finest ground cement and, other things being equal, that most suitable for concrete, may show the poorest results. The greater proportion of very fine flour gives the better cement a tendency to swell, whereas, a cement containing less fine flour and more inert clinker might show greater constancy of volume.

Cements that do not show up well in neat tests for constancy of volume may be quite sound in mortar or concrete, though of course the cause of swelling in a test may be the presence of free lime or magnesia and may have no relation to the fineness of grinding.

Lime.

Lime, while it is not ordinarily used in making concrete, is of importance because of its use in mortar and because it may be made use of in decreasing the permeability of cement concrete, also because it is one of the constituents in the manufacture of slag cement.

Common lime is made by roasting or burning limestone, or carbonate of calcium. The roasting reduces the carbonate to oxide of calcium or quick-lime. Quick-lime usually contains from 5 to 10 per cent. of impurities. The impurities retard the process of slaking, so that lime should be slaked several days before it is used in mortar, so as to be thoroughly slaked when placed in the wall. The swelling of lime in slaking would be detrimental to the strength of the wall. Lime may be slaked and packed in barrels for an indefinite length of time, or it may be kept in the mixing boxes covered with sand. It is when lime mortar is exposed to the air and absorbs carbon di-oxide therefrom that it becomes hardened and that it acts to cement together the bricks or stone of a wall. Kept in the form of a paste, with only the surface of the mass exposed to the air, this absorption of carbon di-oxide will not take place to any great extent. Quick-lime, if exposed to the air, will absorb not only moisture, but carbon di-oxide as well. Air slaked lime will make weak

mortar on account of the premature absorption of the C O 2 and the formation of what is commonly known as whiting. Lime should therefore be fresh-burned.

Neat lime would be of little use as a cementing material on account of its weakness and the necessity of its being in thin layers to enable the air to have access to it. It should be used only to fill the voids in some material such as sand or brick dust.

A barrel of lime will make about eight cubic feet of stiff paste. The weight of a barrel of lime is 230 pounds. This is three bushels or 3.75 cu. ft..

Lime comes in lumps, as it needs no grinding after burning, on account of the fact that the action of the water in slaking breaks up the lumps into a powder. If the lime is found in a powder, it indicates that it is air slaked. The slaking of lime requires about two parts by weight of water to one of lime.

Fat or rich lime is lime made from pure or nearly pure carbonate of calcium. Dolomite is a limestone containing magnesia. Lime made therefrom is poor or meagre, that is, slow in slaking and not fat or rich.

Hydrated lime is a product lately come into extended use for the various purposes for which slaked lime may be used. This is a very fine white powder produced by slaking in a rotary pan quick-lime that has been broken up into small lumps. Sufficient water is used to slake the lime and still leave it hot enough to drive off the surplus water, leaving a dry powder. This powder is sifted or screened to remove unburned or unslaked lime and any coarse particles. The hydrated lime is sold in sacks. This is used in mortars; in the manufacture of sand-lime bricks and of slag cement; in hard wall plasters, either mixed with Portland cement or gypsum products; as an addition to Portland cement mortar to make it more easily worked and to retard the settling; in rendering concrete more dense and waterproof.

The action of lime in the manufacture of what are called sand-lime brick is quite different from its action in or-

dinary mortar. Part of the sand used in making sand-lime brick is very finely ground, and thus rendered active in some such way as the inert clinker from which hydraulic cement is made is rendered active by grinding. The bricks are cured in steam under pressure and the presence of CO_2, instead of being necessary, is undesirable. The chemical action is between the lime and the finely powdered sand.

Sand.

Sand is the term commonly applied to small particles of quartz; it is also used to designate small particles of stone such as crusher dust or very small gravel. The maximum size of grains admitted under the term sand is sometimes as large as ¼ in., while sometimes finer sand is required. Ordinarily about one-sixteenth of an inch is the maximum size of grain.

Standard sand, used in making mortar tests, is crushed quartz that passes a sieve with 20 meshes to the inch and is retained on a sieve of 30 meshes to the inch.

Sand with angular grains is called sharp sand. Bank sand is usually a sharp sand, while river and sea sand generally have rounded grains on account of the wearing of the particles on each other. Formerly specifications generally called for sharp sand. The sharpness of sand has been found to have but little relation to its value in cement mortar or in concrete, and sharp sand is not so often demanded. The most important quality in sand is the grading of the sizes of grains from coarse to fine.

If spheres of equal size and having radii of unity be stacked so as to have the least percentage of voids, each may be considered as enclosed in a solid having 12 faces, each of which is a rhombus whose long diagonal is 2 and whose short diagonal is the square root of 2. The volume of the sphere is 74.05 per cent. of that of the solid. The voids, therefore, in a stack of spheres placed as compactly as possible would be nearly 26 per cent. As sand

is composed of more or less rounded grains, we would look for a percentage of voids approaching this. However, on account of the friction it is difficult to compact sand of grains sifted to a uniform size to a density showing less than about 44 per cent. of voids. It is found that the voids in compacted sand of varying sized grains amount to about 30 per cent. The percentage in ordinary sand varies between 30 and 45.

In general sand having a large sized maximum grain, that is, coarse sand, is a stronger sand for mortar or concrete than fine sand. Many fine sands are unfit for use in mortar and concrete. Such sands could be improved by the addition of a coarse sand, using a quantity of each in the mixture for mortar or concrete. By making several mixtures and comparing tests therefrom with a standard a satisfactory sand may be obtained, and fine sand that might be available and cheap could, by the purchasing of some coarse sand, be made use of to advantage. If only fine sand is available, its weakness may be counteracted by the use of a larger proportion of cement.

Sand containing much mica is apt to be weak, as mica will not adhere well to the cement.

Sand should not contain much foreign substance of a soft nature, such as clay or silt. If this foreign substance is in the shape of lumps, it is especially bad, as these will make weak spots in the concrete. Lumps should be eliminated by sifting in preference to washing. Fine particles of clay in sand, found naturally in the same and thoroughly mixed through the mass, do little or no harm. Clay has been found by some experimenters to reduce the strength and by others to increase it. The reason for this discrepency is no doubt found in the difference in the nature of the clay. If the clay is added to the sand, the probabilities are that it will weaken the mortar, unless the clay is first finely ground. If the clay is found in the sand naturally, it will probably add to the strength, because it is then more apt to be in a finely ground state. It is only *when the clay is* in the finest kind of a powder,

or, if moist, in a finely divided state, and thoroughly mixed through the mass that it has a beneficial effect on the mortar.

Many tests have shown sand containing 5 to 10 per cent. or more of fine clay to be stronger in a mortar than clean sand. A little clay increases the density of the mortar. Sand containing more than 10 or 15 per cent. of clay or silt should be used only after thorough tests and not at all on such work as reinforced concrete. The ultimate effect of the clay on the life of the concrete is a matter of doubt, even though short time tests do show an increase of strength because of its presence. In massive concrete the sand may contain 10 to 15 per cent. of clay and still be suitable. In reinforced concrete it should contain no more than 5 or 10 per cent. In concrete blocks no more than 5 per cent. should be allowed. The richer the mixture, in general, the greater the detrimental effect of foreign substances in the sand. Clay is less harmfull in coarse sand than in fine sand. It helps to fill the voids in coarse sand, while it holds the particles of sand apart in fine sand.

If sand is found to contain an excess of clay, it can be washed to remove the clay. Washing, however, besides being expensive is liable to take out the fine grains of sand, which are of great importance, as these fill the small voids and make the concrete dense and strong. The washing of sand may be done by use of an inclined trough with a gate at the low end. Sand is placed at the high end and played upon with a hose. The clean sand settles at the low end of the trough, and the dirty water flows over the gate. Sand should be tested, washed and unwashed, to determine whether washing is profitable.

If sand is ignited, clay will be broken up, and to some extent burnt, and thus rendered less harmful on account of being more durable. This, too, would be an expensive operation.

Dirt in sand is to be viewed with suspicion. Natural

sands are seldom found mixed with soil, unless it be for a small depth on the top of the bed. The dirt may have been incorporated into the sand as a result of handling. At the site the sand should not be laid in the dirt, as the last of it, when shoveled up may contain an excessive amount of dirt and be the cause of a weak batch of concrete.

Good clean sand may be mixed with sand containing an undesirable amount of clay and the mixture made acceptable by this means. This would be preferable, in some cases to washing and losing the fine particles.

The best material for good strong concrete is coarse, clean sand.

Sea sand contains alkaline salts which induce efflorescence. These should be dissolved out with fresh water.

Stone dust or screenings are sometimes found superior to sand in mortar tests.

Dry sand weighs ordinarily from 90 to 100 lbs. per cu. ft. A common weight assumed for ordinary sand is 2600 lbs. per cu. yd. Sand having very large and very small grains may weigh as much as 117 lbs. per cu. ft. or more than 3000 lbs. per cu. yd.

The specific gravity of quartz is 2.65, hence a solid block of one cu. ft. would weigh 165 lbs. The voids in quartz sand may be calculated from the weight per cu. ft. by noting the per cent. that it falls short of 165. Pure sand weighing 100 lbs. per cu. ft. has close to 40 per cent. of voids.

Besides its use as a constituent of concrete and mortar sand is used in casting artificial stone blocks. For this purpose the sand is made use of just as in a foundry. It is found to absorb the surplus water in the mixture, and this keeps the mold moist.

Aggregates.

The term aggregate is used by the author to mean the solid inert part of concrete not included in the term sand, and not, as used by some writers, to include the sand. This is the rational use of the term. There is need of a single word to apply to the third term in the ratio of a concrete mixture, on account of the variety of substances that may be employed. A 1:2:4 concrete means in America a mixture of one part, by volume, of cement to two parts of sand (whether this be siliceous sand or rock screenings) to four parts of aggregate. This use of the term is very common and convenient. Anyone who would force a different meaning to a term in common use, because of fancied perversion of its true meaning, is commended to the work of restoring the word manufactured to its true meaning, "hand made."

The aggregates used in concrete are usually gravel, broken stone, and cinders. Besides these shells, broken bricks, and slag are sometimes employed.

Gravel, when it comes in assorted sizes, is a very good material for concrete from several standpoints. First, it is usually hard and durable stone that has withstood attrition, and is the result of natural selection. Second, the stones, being round, will, if graded in size, make a dense mixture, and the denser the mixture of a given material the stronger the concrete. Third, gravel will resist fire better than many other kinds of stone. The round, smooth surface of gravel is not as good a surface for the cement to take hold of, and for this reason broken stone concrete may be stronger, though it may not be as dense as gravel concrete. However, the density of gravel often overcomes this, and gravel concrete is sometimes stronger than stone concrete. Some experimenters have found gravel to be stronger than broken stone in concrete. The surface of gravel concrete cannot be tooled as well as that of broken stone concrete, because the *gravel stones* break out under the force of the tool.

Crushed gravel is sometimes used in concrete. This is a very good material, if the gravel is a good clean variety.

Gravel is apt to be mixed with dirt, either in the form of lumps of coal or of slimy covering on the stones, or, as in the case of sand, in the form of an admixture of clay, mud, or silt. Such gravel should be washed or rejected. The objection to the washing of sand does not apply with the same force to gravel, as the gravel is not required to contain the very fine particles that should be a constituent of all good sand.

Broken stone should be hard and durable. Concrete cannot be strong if made of weak stone. The stones should not have incipient cracks, so that they will crush under the rammer. They are best to be nearly cubical, at least not in flat flakes, as such stones will not pack well. The stone should be uniform, that is; there should not be just a few of the largest size of stones, or an excess of the same; also there should not be an excess of dust, say not over 15 per cent. of very small particles. The whole pile, as well as different deliveries should be well mixed, so that different batches of concrete will be as near alike as possible. There should not be over one per cent. of rotten stone. Unloading of the stone is apt to separate the large stones from the small ones. Dumping in dirty places may result in a large quantity of dirt getting into a batch of concrete.

The kinds of stone generally used are trap, limestone, and sandstone. Trap is the best of these, as it is harder and stronger than the others and is a better fire resistant. Hard limestone is good and durable, except as to its ability to withstand fire. It will calcine or turn to lime under heat. Sandstone is not as strong as the others.

In an aggregate such as gravel and broken stone used in concrete strength and density are desired. These are closely related. The mixture that will pack the closest will give the strongest concrete in a given quality of stone. The *voids in broken* stone usually run from 40 to

50 per cent. of the volume. These voids must be filled up with mortar, or the mixture of sand and cement in the concrete. The smaller the amount of voids the better will be the concrete. Tests have shown the unexpected in the matter of the sizes of the broken stone or gravel, namely; that the mixture that has the largest maximum size of stones, if the sizes are properly graded, will be the densest and strongest. Many specifications, particularly earlier ones, read as though it were a serious fault to have sizes of stones too large to pass through a given sized "ring," and many of these same specifications require also that screenings be rejected from the mixture. Both of these requirements are detrimental to the strength of the concrete. A mixture in which the medium size of stones predominates will not be a dense mixture; the exclusion of large stones and very small ones tends to produce this very condition. It is well known that stones of large size have less surface for a given weight, hence there is less surface to cover with cement in concrete of large aggregates, besides less voids to fill. However, there are practical reasons for limiting the maximum size of stones in most concrete work. Some of these are the danger of air spaces forming under large stones and of their arching and leaving voids not filled with mortar.

Given an aggregate having a certain maximum size of stones, the densest and best mixture will be found when about one-third of the weight is composed of pieces less than one-tenth of the maximum dimension, and one-third is composed of pieces between one-tenth and one-half of the maximum, and one-third is composed of pieces more than one-half of the maximum. For example, suppose a mixture has stones that will measure not over one-inch maximum size. One-third of this mixture should pass through a sieve having 10 meshes to the inch, and two-thirds of the total should pass through a sieve having two meshes to the inch. There should be graded *sizes in* all the aggregate, from the largest to the smallest

size, to fill the graded sizes of voids that will of necessity occur.

In general, in a concrete mixture, an aggregate having a large maximum size of stones demands a sand of large maximum size of grains, while an aggregate having a smaller maximum size of stones should have a sand of small maximum size of grains. Thus, if the maximum size of stones is 2½ in., the maximum size of sand grains may be ¼ in., and if the maximum size of stones is ½ in., that of the grains of sand may be 1/16 in. This consistency should be observed in order to have graded sizes in the entire mixture.

Rock dust was formerly considered objectionable in an aggregate. It is now generally recognized that this is very valuable in concrete, as it fills the smaller voids and makes the concrete more dense. Of course, if this dust is rotten stone ground up, it should be screened out. If the rock dust is not uniformly distributed it may be best to screen it out and use it as sand.

The aggregate for small work should have small sizes of stone. In reinforced concrete beams and columns the maximum size of stones should generally be ¾ in. to 1 in. In arches a good size, if no small meshes are employed in the steel reinforcement, is 2 in. to 2½ in. The reason that the stones should be small is so that they will pack around the reinforcement and not leave voids. In walls and heavy work large sized stones are not objectionable. Rubble concrete, sometimes called cyclopean concrete, is concrete in which large stones are embedded. These may be almost any size convenient to handle. They should not be placed near the surface or close to each other. Rubble concrete is used in massive work. In some concrete work large boulders are placed against the forms for appearance, to give a rustic look.

The weight of broken limestone per cu. ft. is about 85 to 95 lb. or about 2300 to 2600 lb. per cu. yd. Gravel and broken trap weigh 2800 to 3000 lb. per cu. yd.

Cinders are often made use of in concrete. Cinder concrete is very useful in filling in between the sleepers in buildings. It could also be made use of in filling in the spandrel of a stone or concrete arch. If the cinders are well burned and free from lumps of coal, cinder concrete makes an excellent fire protection. It is light in weight and porous and a very poor conductor of heat. It has the further advantage that nails can be driven into it with ease for a month or two after it has set. Cinder concrete is not very strong. Cinders usually contain much sulphur. Steel laid in cinders, if moisture be present will rapidly corrode, on account of the formation of sulphuric acid, due to oxidation of the sulphur and addition of water. Iron or steel pipe laid in a cinder fill should be surrounded by concrete or at least by clay. Cinder concrete 1:2:4, mixed wet, is probably as good a protection for steel as stone concrete. Cinders for concrete should be clean. It is sometimes necessary to wash them and to sift out the ashes as well as to break up the large clinkers.

In some localities there are large deposits of shells, which, mixed with sand and cement, make a good concrete. Tests of any such substances should be made to determine the best mixture and the strength.

Slag is used to some extent in making concrete. Ordinary broken slag does not seem to be a desirable aggregate on account of its unstable chemical composition. This is especially true of fresh slag. Seasoning may dissolve out the undesirable chemical compounds. When molten slag is run into water, it is granulated, and much of the objectionable sulphur is washed out. Pulverized and mixed with slaked lime alone this granulated slag can be made into bricks or blocks.

Crushed marble is employed in the manufacture of some brands of artificial stone. This may be mixed with cement alone.

Mortar.

Mortar may be lime and sand, neat cement, cement and sand, or cement, lime and sand.

Lime mortar is made of one part of lime paste to about 3 or 4 parts of sand. The New York Building Code gives 1:4 as the proportion. The volume of the resulting mortar is about equal to that of the sand. Common lime mortar will not set in water. It takes a long time for it to set in a damp place. If air is excluded, it will never set. Nevertheless some water is required in the setting process, and lime mortar suddenly dried will be killed. Hence it is well to moisten bricks before they are laid in lime mortar, especially if they are porous. Lime mortar should not be used in very thick walls or in very thick joints; also it is unsuitable for making concrete. The reason for this is twofold: in the first place air must have access to the mortar to harden it; in the second place the small tensile strength of the mortar makes it weak to resist lateral flow, and it is consequently of little use in overcoming compression in a mass. The ultimate tensile strength of lime mortar is about 20 to 50 lbs. per sq. in. The compressive strength depends largely upon the thinness of the joints. Cubes of mortar, when tested in compression stand but little; but, when the sand is confined in joints between bricks or stone, the adhesive strength of the cementing lime is taxed less, hence a greater crushing strength can be withstood. The report of tests made in 1884 at the Watertown Arsenal show cubes of lime mortar to stand 120 lbs. per sq. in. in crushing. The same report shows brick piers built with lime mortar to stand from about 1000 lbs. to 2000 lbs. per sq. in. The strength of a wall laid in lime mortar will be gaged not so much by the strength of the brick as by that of the mortar, since lime mortar, even confined in joints, is only about one-tenth to one-fifth as strong as ordinary brick. Lime mortar requires about a month to set sufficiently to receive any considerable load, but it continues to harden

indefinitely. In plasters, gypsum or plaster of Paris is often added to hasten the setting. Dry and thoroughly set lime mortar weighs about 110 lbs. per cu. ft.

Lime mortar is greatly improved by adding cement, either Rosendale or Portland. The strength is increased, and the time of setting is shortened. On the other hand cement mortar for brick or stone work is improved by the addition of lime. It is made easier to work with the trowel; it does not set up so quickly, giving more time to place it; the adhesive value is improved on such surfaces as concrete blocks. A common mixture is that of equal volumes of 1 to 3 lime mortar and 1 to 3 cement mortar. The mixture of the two mortars is called cement and lime mortar. Lime paste or hydrated lime may be added to cement mortar without being first mixed with sand. The addition of 10 to 25 per cent. of lime paste to cement mortar will decrease its permeability and makes it more adherent to old concrete and to concrete blocks. Lime paste is used in making some artificial stone, of sand and cement, to increase the density.

The addition of 10 to 20 per cent. of hydrated lime does not destroy the hydraulic property of Portland cement. Cement mortar to which lime is added will attain greater strength when stored in water, after a day or two in air, than when kept in air only. Mr. H. B. Nichols, in a letter in Eng. News, Dec. 5, 1907, describes some tests on adding lime paste to 1:3 Portland cement mortar and 1:2 natural cement mortar. It was found that from 4 to 20 per cent. of lime paste added to natural cement mortar increased the strength on an average 25 per cent. Percentages of lime below 20 did not seriously weaken Portland cement mortar; greater percentages did weaken it. The briquettes required to be kept in air 48 hours before being stored in water. Lime increased the adhesion of mortar very materially.

Tests from the Government report above referred to for blocks of cement and lime mortar, one part cement mortar and two of lime mortar, showed a strength in

compression of 175 to 200 lbs. per sq. in., Portland and Rosendale cement showing about equal strength. Brick piers in similar mortar showed a compressive strength of about 1500 lbs. per sq. in.

Cement without sand, or neat cement, is not very often made use of. It is, however, useful in setting anchor bolts in drilled holes in stone work and is considered better than lead or sulphur for this purpose. The ultimate adhesive strength is about 400 to 500 lbs. per sq. in. of surface of bolt in contact with cement. Blocks of neat Portland cement about 2 years old showed a compressive strength of about 5000 lbs. per sq. in. Broad, flat blocks showed a strength two or three times as great (Govt. Report, 1884), showing that in joints between bricks the crushing strength of neat Portland cement is ten to fifteen thousand pounds per sq. in., or about the strength of good hard brick.

A series of tests that serve to show not only the tensile strength of neat cement of the average American brand but also the quality of the cement and the strength of 1:3 mortar is that made in 1900 by the Dept. of Public Works of Philadelphia on 8 different brands of American Portland cement. In all over 4000 tests were made. The averages of the results from the different brands are as follows:—

99.4% passed through a No. 50 sieve.
90.3% passed through a No. 100 sieve.
75.6% passed through a No. 200 sieve.
Specific gravity, 3.124.
Time of initial set, 73.6 minutes.
Time of hard set, 358.2 minutes.
Rise of temp. of paste in setting, 5.4 degrees.

Ultimate Tensile Strength.	Neat.	Pounds per Square Inch. 1:3 Std. Quartz Sand
1 day	449	88
7 days	724	234
28 days	792	311
2 mos.	792	323
3 mos.	803	327
4 mos.	831	329
6 mos.	818	333

A later summary of average results of cement tests made in the Philadelphia laboratories is the following given by Mr. E. S. Larned in a paper read before the A. A. P. C. M., Atlantic City, N. J. Sept. 1907.

Proportions	Tensile Strength, lb. per sq. in.					
	7 days	28 days	2 mos.	3 mos.	4 mos.	6 mos.	12 mos.
Neat cement	710	768	760	740	732	758	768
1 to 1 mortar	590	692	690	680	680	685	695
1 to 2 mortar	370	458	460	455	453	458	460
1 to 3 mortar	208	300	310	310	310	310	308
1 to 4 mortar	130	210	230	230	230	232	232
1 to 5 mortar	80	150	185	195	195	195	197

The above tabulation was interpolated from the diagram of cement mortar tests prepared by Mr. W. Purves Taylor. The results of the neat tests and the 1:3 mortar tests (i. e. one part cement to three parts crushed quartz, by weight) are averaged from over 100,000 tests, while the other results are based on from 300 to 500 tests. (See Concrete, Oct. 1907.)

One volume of Portland cement, measured loose, mixed with one-third of a volume of water will shrink to .78 of a volume of stiff cement paste. A volume of Rosendale cement and .4 of a volume of water will make about the same amount of paste as the Portland. It takes 6.75 bbl. of cement to make one cu. yd., measured loose, hence

a cu. yd. of cement paste would require 8.65 bbl. of cement. This is on the basis of 4 cu. ft to a barrel of cement. A bag or quarter barrel of cement is accepted by many engineers as a cubic foot. Sample bags should be measured and weighed to see that this holds true.

The office of cement in mortar is to fill the voids in the sand and at the same time cement the particles together. The voids in sand are about one-third of the volume. A mixture of one part of cement to three of sand will therefore give a correct proportion on this basis and result in a volume of mortar not much if any, more than that of the sand. This is the common mixture for cement mortar. The richer the mortar in cement the greater will be its tensile strength. The compressive strength does not increase in the same proportion, but depends somewhat on the thinness of the joints. For brick work or cut stone work there is no advantage in using richer mortar than 1:3 if the wall is to be in compression only. In concrete or rubble work richer mortar by its greater tensile strength prevents lateral failure under compressive stresses.

Richer mortar than 1:3 will occupy more space than the volume of the sand. One volume of average sand can be estimated to give one volume of 1:3 mortar; 0.9 volume of sand can be estimated to give one volume of 1:2 mortar; 0.7 volume of sand will give one volume of 1:1 mortar. One cu. yd. of 1:3 mortar will require about 2¼ bbl. of Portland cement; one cu. yd. of 1:2 mortar will require about 3 bbl. of cement; one cu. yd. of 1:1 mortar will require about 4.7 bbl. of cement.

Mortars are generally mixed by volume in actual construction. In laboratory tests, for greater accuracy, the quantities are usually measured by weight.

Brick work with ⅛″ joints requires about one-eighth of the volume in mortar. For ⅝″ to ½″ joints about 20 per cent. will be mortar. Ashlar masonry requires about 6 to 8 per cent. of the volume in mortar, and rubble masonry requires 25 to 40 per cent. of mortar.

The amount of water required is about one-fourth the volume of the cement in neat cement or about one-half of the volume of the cement in sand and cement mortar.

Portland and Rosendale cements may be mixed in any proportion in mortar. If equal quantities of each be used, the strength will be about the mean between that of the separate mortars, but it will set in about the time required for Rosendale cement.

As in the case of lime mortar, mortar made of cement and sand will stand much greater crushing load in the thin joints of a brick wall than in cubes. The strength in the thin joint is sometimes many times as much as the same mortar in a cube. Good 1:3 Portland cement mortar in thin joints will generally withstand the pressure that will crush the strongest brick or stone.

Mortar made with crusher dust, that is, stone dust that results in the crushing of stone, was found by Government tests to be superior in tensile strength both to sand and crushed quartz. In the report of the Chief of Engineers, U. S. A., for 1902 (See Eng. News, Apr. 2, 1903) the following results of tests are shown.

First Series of Tests.

No. of Sand	No. of Tests, Each Period	Age 24 hrs.	7 da.	1 mo.	3 mo.	6 mo.	1 yr.
				1:3			
1	14	103	370	397	376	381	355
2	36	105	241	274	294	290	291
3	12	79	397	544	607	628	602
				1:4			
1	14	53	243	282	266	259	227
2	36	65	169	198	207	192	185
3	12	37	267	395	494	512	484
				1:5			
1	14	31	187	221	211	190	161
2	36	36	132	155	159	145	144
3	12	25	211	336	428	421	416

Second Series of Tests.

No. of Sand	No. of Tests, Each Period	24 hrs.	7 da.	1 mo.	3 mo.	6 mo.
				1:3 Age		
1	12	62	302	425	449	436
2	12	58	224	289	310	310
3	12	111	399	505	603	593
				1:4		
1	12	17	153	283	329	325
2	12	23	136	193	220	234
3	12	60	287	411	453	501
				1:5		
1	12	9	103	189	244	263
2	12	7	85	130	168	184
3	12	34	212	322	377	429

Notes. No. 1, standard crushed quartz; No. 2, Plum Island sand; No. 3, crusher dust. Proportions are of Portland cement to sand by volume. In first series of tests mortar was quite wet; in second series consistency was medium. Values in table are ultimate tensile strength in lbs. per sq. in.

Some tests given by Mr. G. J. Griesenauer in Eng. News, Apr. 16, 1903 show limestone screenings to be superior to sand in bricquettes tested in various periods from seven days to one year. Bricquettes made of screenings as lean as 1:3 showed greater tensile strength at the end of one year, in some cases, than neat Portland cement.

Unlike lime mortar cement mortar will set in fresh water or sea water or entirely excluded from air or water, except the water used in mixing. Some substances, however, will cause cement mortar or concrete to disintegrate while setting. Water containing the discharge from a pulp mill was found in one case to cause setting concrete to become absolutely worthless. (See Eng. News, Feb. 5, 1903). Manure, especially if it be wet, will rot concrete while it is setting, but does not seem to affect concrete that is set *and hardened.* (See Eng. News, Jan. 1, 1903.

p. 11; Jan. 29, 1903, p. 104; Feb. 5, 1903, p. 127). Manure is sometimes used and recommended to prevent concrete from freezing. This is bad practice, if the manure is placed in contact with the concrete. It is not only liable to rot setting concrete, but it will discolor it. Weak acids will attack the lime in mortar or concrete that is setting.

The effect of oil on concrete is not well understood. In Eng. News, Vol. 53, p. 279 and Vol. 58, p. 16, some tests are reported which show that most oils have a deleterious effect on concrete. Concrete or mortar that is setting is especially susceptible to the weakening influence of oils. Animal and vegetable fats and oils are the most harmful, excepting such drying oils as boiled linseed. This latter, while it penetrates concrete to some extent does not appear to weaken it. Petroleum does not seem to injure concrete. Mr. H. T. Poe, Jr. in Engineering Record, Vol. 55, p. 222 tells of a reservoir painted inside with coal tar which was used for fuel oil and gave satisfaction. An experimental tank of 1:2:4 concrete, not treated, with fuel oil in 9 months showed no disintegration and no leaks, only discoloring of concrete for 1½ in. (Eng. Record, Vol. 55, p. 9.) Tanks not treated were also found to be unaffected by petroleum and petroleum products by Mr. J. L. Gray (Eng. Record, Vol. 55, p. 313.) See Eng. News, Vol. 57, p. 13, where test of 1:2:4 concrete tank filled with fuel oil gave satisfactory results. Oil penetrated concrete less than ½ in. See also Eng. News, Vol. 53, p. 279, and Eng. Record, Vol. 51, p. 357.

Alternate freezing and thawing during setting destroys weaker mortars, such as lime mortar or Rosendale cement mortar. Some experimenters have reported that it does not affect the strength of Portland cement mortar. It is the general opinion, however, that it is harmful to any setting mortar or concrete. Freezing delays the setting of mortar, but if thawing does not occur until the mortar is set, it does not seem to weaken it. The setting of mortar is sometimes entirely suspended due to its being frozen. Upon thawing the setting resumes. Concrete in

ndations has been found, after having stood for some e in freezing weather, to quake; then, after having e to set in warmer temperature, it has become good . hard. Freezing is particularly harmful when there , mortar finish on concrete poorer in cement, or where facing is a richer mixture than the body of a wall. e rich mortar is apt to break off from the main body. ck walls in mortar should not be run up too fast in ezing weather. The heavy load coming upon the lower ts of walls thus built have caused many failures on ount of the fact that the mortar had frozen and not When the frozen mortar thawed it was almost in the dition of fresh mortar. Failures have also occurred concrete work on account of the fact that centers were noved from work that was frozen and not set.

n a paper by Messrs. P. L. Barker and H. A. Seymonds, lished in Eng. News, May 2, 1895, a number of tests given on the strength of cement mortars subjected to ezing temperature. These experimenters found that tland cement mortar suffers no surface disintegration ler any condition of freezing, but the strength is diished, sometimes very materially; Rosendale cement rtar disintegrates on the surface, but seems to acquire ater strength in the part not disintegrated; the cohesion Rosendale cement mortar is destroyed by immersion water which freezes around it; salt used in mixing ter (about 7 per cent.) helps Rosendale cement morto resist surface disintegration, but appears to diminthe strength; a mixture of Portland and Rosendale nents was found to combine the good points of each, nely, to resist surface disintegration and not to lose its ength due to freezing; lime mortar kept frozen until had time enough to set was not injured, but when alnately frozen and thawed it disintegrated.

Salt in the water is sometimes used to prevent freezing mortar. On exposed walls salt is liable to produce oresence and to disfigure the wall. Moist salt is own to corrode steel, so that concrete in which steel is

embedded, if it contains salt and becomes wet, may corrode the steel. Whether or not the steel is safe from corrosion if the concrete is kept dry is a matter of uncertainty. Heavy walls in plain concrete do not need salt in the mixture, unless it be in extremely low temperature, for the cement in setting generates heat. Salt may be found useful in such thin unreinforced work as sidewalks. The use of such substances as salt, sugar, soft soap, etc., sometimes resorted to for various purposes, seems to be of doubtful benefit. At least the effect of these substances is not clearly understood.

Heating the materials of mortar or concrete before mixing is very often practiced and is very often recommended to prevent freezing. Experiments have shown that it is harmful to use hot or even warm materials. Mr. Wm. M. Maclay, in Trans. Am. Soc. C. E., Vol. VI, 1877, describes tests in which the heating of ingredients of cement mortar to 100 deg. F. reduced the strength to only 7 to 30 per cent. of that of specimens made at 40 deg. F. Experiments made by the Austrian Society of Engineers and Architects (See Eng. News, Vol. 31, p. 253) led to the conclusion that mortar mixed with warm water shows about the same deterioration in freezing temperature as when cold water is used.

The only warrant for the use of hot materials seems to be the very doubtful one that it is often done. The only evidence that it is not harmful as practiced appears to be the negative evidence that no failures have been traced to the use of hot materials. Heat increases the activity of cement, and quick setting of mortar, that is, setting during the process of placing, is detrimental to its strength. Heat drives off water the presence of which is necessary to the proper hardening of the cement. Concrete bricks or blocks are sometimes steam cured, but in this case, the heat is applied after the concrete is placed in the forms and not before it is handled in a plastic state. The steam immersion would not take away water from the concrete, but would rather add to that already there by

condensation, unless the blocks were heated by other means hotter than the steam. In doing concrete work in hot weather it is desirable to keep the materials cool and the concrete protected from the direct rays of the sun. These adverse conditions should not be artificially produced in cold weather.

The safest and best materials for concrete are good, clean, hard broken stone or gravel, good clean coarse sand, finely ground Portland cement of known quality, and good clean fresh water. The best temperature for these materials is as cool as they can be kept without being below freezing.

As stated, water for use in making mortar or concrete should be clean. It should not be water from coal mines. It should not contain sewage or rotting substances. It should not contain waste from pulp mills and should not be acid in reaction. Sea water for concrete is doubtful. It should not be used, if fresh water can be obtained.

The mixing of mortar should be thorough. If done by hand, the sand should be placed on the mixing board and spread out to an even thickness of about 2 or 3 inches. Upon this the cement should be evenly spread and the dry materials turned over three or four times with shovels. Water is then added. It should not be dashed on so as to wash away the cement. After the addition of the water the mortar should be turned until the pile is uniform throughout both in color and in consistency.

Machine mixing, both of mortar and concrete, generally results in a more uniform product than hand mixing. There are two general classes of mixers, namely, batch mixers and continuous mixers. In the batch mixer the materials for one-half to one cubic yard are put in at once, and this is turned over until all are uniformly mixed, then the whole is discharged. In a continuous mixer the various materials are fed regularly into the mixer, and concrete is discharged continuously.

Thorough mixing and uniformity of product are the prime essentials. There should be in any given quantity,

say a shovelful or a wheelbarrow full, just the amount called for of the several materials. If 1:2:4 concrete is called for, there should be one part of cement, two parts of sand, and four parts of broken stone or gravel in each such quantity throughout. This is true of the amount of water used as well as other ingredients. Each batch and every part of the batch should be of the same consistency. If this thoroughness and uniformity are not maintained, the resulting concrete will not be homogeneous.

In a batch mixer it is not difficult to secure uniformity as to amounts of ingredients in each batch, if a careful man is placed in charge. There are, however, practical difficulties in measuring the materials. One of these is the fact that the most convenient thing to haul sand and stone from the pile is a wheelbarrow, and it is very difficult to gage the capacity of a wheelbarrow or to get workmen to load one the same amount at each trip.

It is quite evident that to make three or four different streams discharge proportionate amounts of materials, as must be done in a continuous mixer, is fraught with many difficulties and difficulties that are harder to overcome than the measuring of material before they are put into a batch mixer. Continual vigilance is necessary in order to make sure that none of the various streams become clogged or fail to discharge their proper amounts of material.

Cement mortar should not be mixed in large batches, if it will require long to use up the batch. It should be fresh mixed and used as soon after mixing as possible, as it begins to take on its initial set in a short time. If it is necessary to leave mortar stand a while, it should be re-tempered, adding a little water if necessary, before it is used. It is said that re-tempered mortar will adhere better to old concrete surfaces or to concrete blocks than fresh mortar. For the same reason re-tempered concrete might be found useful in joining to old work or to concrete that has partially set.

Re-tempering of mortar and concrete are usually prohibited because of the uncertainities attending the practice.

Rosendale cement mortar and concrete are injured to a greater extent by re-tempering or by delay in placing than Portland cement mortar and concrete. This is because Rosendale cement takes its initial set in a shorter time than Portland. Experiments reported by Mr. Thos. S. Clark and published in Eng. Record, Dec. 27, 1902, show that Rosendale cement, neat and with sand, if re-tempered by adding water, after allowing to stand for one hour, is reduced in tensile strength about 50 per cent. for long-time tests. For tests which set but a few days greater weakness than this was shown for neat cement though not so much difference for tests with sand. The same report states that no marked difference was shown in the case of Portland cement re-tempered after one hour. It is probable, however, that Portland cement re-tempered after several hours standing would show similar weakness.

Continuous mixing of cement mortar for several hours seems to cause it to retain its life, that is; it seems to retard its setting. Some experiments reported by Mr. G. Y. Skeels, in Eng. News, Nov. 6, 1902, show that the tensile strength of Portland cement bricquettes of 15 days standing, after continuous mixing for 9 or 10 hours was reduced only about one-quarter from the freshly mixed cement. Similar results were found both for neat cement and 1 to 2 mortar.

Concrete deposited under water has been found to be better if allowed to stand a few hours after mixing, as the cement will not leech out so readily.

One part of brick dust, one part of quick lime, and two parts of sand, mixed dry and tempered with water will make a hydraulic mortar.

Grout is the name given to thin or liquid mortar. It is used for filling in joints or spaces which cannot otherwise be reached. Cast bases for columns are usually left

rough on the bottom surface, and holes are provided in the bottom plate for grouting. The grout is prevented from running away by banking up sand around the edge of the base. Grout may be used to strengthen walls in which the mortar has washed out. It may also be used in making concrete by first placing broken stone or gravel and then pouring the grout into the interstices. A fill of broken stone or a gravel bed in which there is little or no sand may be consolidated by filling the voids with grout.

The method referred to in the last paragraph of pouring grout under the base of a column may be used to advantage also for bridge shoes. It is a good plan to wedge up a bridge shoe an inch or two above the surrounding masonry (allowance being made for this in the plans), and then after having built a dam of sand around the edge, to fill in under the shoe with grout. This keeps water from lying around the bridge shoe.

In Eng. News, Aug. 8, 1907, p. 145, there is a short description of repairs made on a bridge pier in Germany that had been cracked horizontally below the water level and displaced by the action of an ore steamer. The outer edges of the crack were sealed by means of wooden wedges and oakum, and in addition a strip of canvas was placed around the pier and bound thereto. Grout was pumped into the crack, by means of air pressure and grouting drums, through pipes provided for this purpose. The grout was a 1 to 1 mixture of Portland cement and sand, mixed dry and then tempered with an equal volume of water.

Concrete.

Concrete is a mixture of mortar and some aggregate, hence what has been said concering mortar and its characteristics has application in a large measure to concrete. The mortar with its contained cement, acts to hold the aggregate together making of the whole an artificial stone. An ideal concrete is one in which the cement fills or more

than fills the voids in the sand to form the mortar and the mortar fills the voids in the broken stone or other aggregate. Proportions in this country are nearly always given in bulk or volume.

The voids in broken stone are about 40 to 50 per cent. of the bulk. Concrete composed of one part of mortar to two of broken stone would therefore be about right. The volume will be practically equal to that of the broken stone. A common mixture for walls and other heavy work is one part of cement, 3 parts of sand, and 6 parts of broken stone. For reinforced concrete a richer mortar is desirable, because of the fact that the covering of the steel with cement must be assured, and because the smaller size of broken stone demands more cement to cover the parts of the aggregate. The standard mixture for reinforced concrete is 1 part of Portland cement, 2 parts of sand, and 4 parts of broken stone or gravel.

Leaner mixtures are sometimes used in heavy work where large sized aggregates can be employed. Such proportions as 1:3:7, 1:4:8, or even 1:5:10 could be used where strength is not an essential characteristic. Included in the volume of the stone there could be large stones, say a foot or so across. These should be separated from each other and from the surface.

In general as few different mixtures of concrete as possible should be specified on one contract, and the line of separation should be well defined, so as to avoid confusion. The body of a pier or abutment could be of 1:3:6 concrete and the coping, because of the need of stronger concrete to take the bridge seats, could be of 1:2:4 concrete. Some steel reinforcement in the coping of such a pier would not be out of place, to tie it together and prevent cracking in the entire pier. It would not be well to call for beams of one mixture and slabs or columns of another, though column footings and columns might be of different mixtures if one were plain and the other reinforced.

In ordinary work it is best to specify standard mixtures for the concrete and to see that the materials are regular, the sand and aggregate well graded in size and uniformly mixed. It would not be discreet to leave to the contractor the determination of the proportion of ingredients. On special work or very large contracts, where the engineer is given discretion, he may determine the mixtures that suit more accurately the materials to be used. The percentage of voids may be ascertained by taking a vessel of known capacity and filling it with the aggregates. Then by weighing it before and after filling level full of water the volume of water may be found from the known weight per cubic foot of water, namely, 62.4 lb. The percentage that this volume of water is of the full capacity of the vessel is the percentage of voids in the aggregate. The same may be done with the sand. By using about 10 per cent. more cement than the voids in the sand and 10 per cent. more sand than the voids in the broken stone a mixture will be effected that is most economical for the given materials.

For quartz sand the voids may be found by weighing a known volume. The specific gravity of quartz is 2.65, and the weight per cu. ft. is 165.4 lbs. If then a cu. ft. of sand weighs say 100 lbs. the difference, or 65.4 lbs. represents the voids. Dividing 65.4 by 165.4 we have 40 as the percentage of voids.

One way that is recommended for finding the proper mixture of sand and stone or sand and gravel is based on the fact that, other things being equal, the denser the mixture the stronger will be the concrete. The operation is that of finding the mixture that has the maximum weight for a given volume. A common galvanized iron pail and spring balances may be used for the purpose. The bucket is filled half full of water, and a batch of the sand and aggregate at a trial proportion is thoroughly mixed and slowly dropped from a shovel. The surplus water is allowed to flow over. Without tamping the bucket is filled level full. The mixture that weighs the

most will give the densest and strongest concrete, and on account of having the least percentage of voids it will require the least amount of cement.

Sometimes concrete is made of cement, sand, gravel, and broken stone, the gravel being intermediate in size between the sand and the broken stone. Good dense concrete may be made of this combination, if suitable proportions are determined.

Concrete is sometimes made of sand and cement only, there being a large quantity of sand in proportion to the cement, say 1 of cement to 7 or 8 of sand. This does not make good concrete. The voids in the sand cannot be filled by so small a proportion of cement.

Concrete may be mixed by hand or by machinery. When hand mixed the following is probably the most approved method. First the sand is dumped on the mixing board and spread out to a thickness of about three inches. Over this the cement is dumped and spread out evenly. This dry sand and cement is then turned over with shovels two or three times, and the pile is leveled off. On this the broken stone or gravel, previously wetted, is dumped and spread out evenly so as to cover the sand and cement. The full amount of water is then measured and poured on. As only the aggregate is exposed, the water may be poured or dashed on without particular care, since the danger of washing out the cement is removed. The whole should now be turned over with shovels two or three times.

When concrete is mixed by machinery, all of the ingredients, including the water, are carefully measured and placed in the mixer. For mixtures in which little water is to be used it may be necessary, in order to get a uniform mixture, to mix the dry ingredients without any water first and then add the water and continue the mixing until this is incorporated in the mass. The number of turns of the mixer, or the time required to effect an *intimate and thorough* mixture will depend somewhat *upon the kind of mixer*. This can be well judged by the

uniformity of the product in color and by inspection to see if the grains of sand and the stones are covered with cement. The whole mass should have the color of the cement. Concrete not well mixed will have patches of bare sand and stones. Regularity both in the proportion of the ingredients and in the amount of mixing are very important in any concrete work and are of special moment in reinforced concrete as well as any work that is to be watertight. When the time of mixing required to give a thorough mixture is ascertained, this length of time should be allowed for each batch.

Usually about 10 or 15 turns of the mixer are required to give a thorough mixture. The time required is from 1 to 1½ minutes. It requires about two to three minutes per batch to put in the materials and take out the concrete, if the handling of materials can be expeditiously done.

Lean mixtures require more mixing than rich ones, because it is more difficult to distribute the smaller amount of cement thoroughly through the mass. Dry mixtures require more mixing than wet ones.

It is well to mold a small cube of concrete from the batch occasionally. These cubes will be useful in gaging the kind of concrete turned out, and by the time required for them to set the hardness of the work may be judged. When concrete is placed in unusual conditions such a sample placed in the same conditions and accessible for inspection furnishes an index to the character of the placed concrete.

Concrete should not be too wet or too dry. If too wet it will shrink an excessive amount from drying out; if too dry when placed, it will not be dense, as the mortar will not run into the spaces. Formerly, when mass concrete was about all the kind made, very dry concrete was generally specified, just enough water being used to satisfy the needs of the cement in setting; and this was heavily rammed. This is not very objectionable in mass concrete, and it has some advantages. One of these is that the drier

concrete will attain its strength sooner than wet concrete. Wet concrete is generally considered to be best for almost all purposes, though the amount of water which should be used is not the same in all cases.

The chief advantage in dry concrete is its faculty of setting up in a shorter time than wet concrete. When a piece of construction is to receive its load soon after placing the concrete, it is best to use a dry mixture, if the conditions are not such as to render such a mixture harmful for other reasons. Some concrete block manufacturers take advantage, in making their blocks, of the fact that dry mixtures hold up in a short time better than wet ones. They use a mixture that is simply moist, so that blocks can be removed from the molds about as soon as they are molded. The result is that the blocks are spongy, and we see houses built of the blocks turn dark gray in a rain because of the absorption of water. Such use of dry concrete can only serve to discredit concrete itself.

Another advantage in dry concrete is that it is less liable to freeze in cold weather than wet concrete. It should be borne in mind that much water in concrete in cold weather renders it more liable to the action of frost. However, other means of preventing freezing should be employed rather than the use of concrete that is too dry for the purpose intended. On the other hand in very warm weather an excess of water is advantageous, because it prevents the concrete from too rapid drying out due to the heat.

Dry or mealy concrete has many disadvantages. It is unfit to use in walls that are to keep out moisture. A mealy concrete wall allows water to flow through it very freely. It is unfit to use in clycopean or rubble concrete, because it will not flow around the large stones and coat them with cement. For similar reasons it is totally unfit for use in reinforced concrete work, as it will not flow around the steel and coat it with cement. Because dry concrete is porous there will be voids around the reinforcing steel as well as in the other parts. The steel is

thus deprived of the protection which the concrete must afford, if the combination is to be lasting. Concrete must be wet to pack around the reinforcing steel or embedded stones and to cover them with cement. Both of these requisites are of prime importance in reinforced concrete. Steel rods will not be gripped as firmly in a porous concrete as in a dense concrete. Dry concrete lacks cohesion, and the cohesion of concrete has much to do with the gripping of the steel. It is of special importance in cinder concrete, reinforced, that a wet concrete be used, so that the cement will coat the steel and protect it from sulphur or other harmful agents in the cinders.

Dry concrete requires tamping; the drier the concrete, generally, the heavier the tamping necessary. Dry concrete for this further reason is unsuitable in reinforced concrete work. Tamping around reinforcing steel is apt to displace the steel as well as to disturb or spring the forms. The concrete around the reinforcement should be worked into the spaces by puddling rather than by tamping, and the consistency of the mixture should be such as to admit of this operation. As near a liquid as possible is evidently the best consistency to effect this end, but other considerations must be taken into account. If concrete is too wet it will shrink an excessive amount on setting and drying. This may give rise to shrinkage cracks or other trouble that such change of volume would naturally lead to.

With concrete too wet, unless the forms are very close, approximating water-tightness, liquid mortar will be wasted through the crevices. This is a fault sometimes met with in concrete work, namely, the mortar being of too watery a consistency, or the forms lacking the proper tightness, not only results in a waste of mortar but leaves the concrete spongy, where the mortar has leaked out.
ay give the appearance of dry concrete, whereas
se is just the opposite.

ıer danger that may attend the use of sloppy con-
ȝ the formation of laitance. This is a milky or

slimy substance that sometimes gathers in the excess water on the surface of concrete. It is composed of about the same ingredients as the cement, but it does not harden as the cement, remaining rather in a gellatinous state. Besides taking from the cement useful elements needed in the concrete, the laitance, if not removed, leaves a film in the concrete where it is not bonded together as it should be.

For reinforced concrete work there must be found a mean between too wet a concrete and too dry a concrete that will meet the conditions. The proper mean is not a mixture medium in constancy, but a very wet mixture. The concrete should assume a nearly level surface. It should flow sluggishly around the reinforcing steel and require little or no tamping. A little shrinking in reinforced concrete is an aid rather than a detriment, because the shrinking of concrete acts to grip the steel.

Other works than reinforced concrete do not generally require such wet mixtures. Work that is to be waterproof or nearly so should be made of a wet mixture and should be puddled, as in the case of reinforced concrete, to work out air bubbles and to solidify the mass.

There should not be an excess of water in the concrete base for a sidewalk, where a richer mortar finish is to be put on subsequently. Tamping will bring this water, and probably accompanying laitance, to the surface and the finish coat will not bond well to it. Pavers usually employ a rather dry mixture for the base and tamp it well. However it is believed that a better pavement would result from the use of a wet mixture and little tamping, with the finish coat floated in at the same time and with as little troweling as possible.

Wet mixtures do not require as much mixing to distribute the cement through the mass as dry ones. However the liquid state of the concrete may make an insufficient amount of mixing appear to be enough; so that just as much vigilance is required to insure the proper mixing.

Dry mixtures are more liable than wet ones to ball in the mixer, that is, to gather in lumps, as the materials will not flow well without the lubricating water. With no water whatever in the mixer there would be no cohesion and the materials would mix, whereas a little water would give a cohesion that may result in the balling referred to. It may be necessary in some cases to mix the materials dry and then add the water giving the mixer a few more turns. While hand mixing in general is less satisfactory than machine mixing, it is possible that in some cases for very dry mixtures hand mixing will produce a better concrete.

As to the strength attained by wet and dry mixtures, it is found that given a number of samples of different consistency the drier mixtures will at first show greater strength. After a few days the mixtures having more water will attain the strength of the drier ones and begin to surpass them. In general the more water used in the mixture the longer it will be in attaining its full strength. Medium and wet mixtures, after many months, reach about the same strength. Very wet and very dry mixtures are both weaker on long time tests than those in which the consistency is not excessive.

The amount of water required in concrete depends upon the porosity of the aggregate, the proportion of the ingredients, the wetness of the sand and aggregate before being brought together, the fineness of the sand, (fine sand requires more water than coarse sand.) and to some extent on the brand of cement. If the sand and stone were thoroughly wet before mixing, the amount of water would depend largely on the amount of cement in a batch, as moistening of these will lessen the amount of water required to be placed in the mixer.

Dry concrete is usually understood to be of the con- ·cy of moist earth. It will retain its shape when ed in the hand. No water will flush to the surface ıping.

Medium concrete is of such consistency that it will not quake in handling and will not quake under light tamping. When well or heavily tamped, the concrete will quake and water will flush to the surface.

Wet concrete will quake in handling and cannot be tamped very much.

In ordinary concrete mixtures dry concrete will require about 5 to 6 per cent. of water (based on total weight of dry materials), medium concrete will require about 6 to 8 per cent. of water, and wet concrete will require about 8 to 10 per cent. This is an average of about 1, 1¼, and 1¾ gallons per cubic foot of concrete for the three respective grades. Per bag of cement it takes about 3 to 4 gallons of water for dry concrete, 4 to 6 for medium, and 6 to 8 for wet. The amount of water should not be specified in any case, unless the proper consistency and the amount of water required therefor have been previously determined by trial with the materials to be used. Some materials will require amounts of water differing considerably from the foregoing.

It is important to wet the aggregate before it is mixed with the other ingredients especially in the drier mixtures and in hand mixed work, and particularly when the aggregate is porous or absorbent. This wetting of the aggregate in the pile allows it time to absorb water that might otherwise be robbed from the cement of the concrete, if the stone is not wet previous to the mixing.

The following table gives approximately the amounts of materials of average quality required to make one cubic yard of concrete of the three most common mixtures.

Proportion	Bbl., P. Cement	Cu. Ft. Sand	Cu. Ft. Stone
1:2:4	1.44	11.5	23.0
1:3:6	1.04	12.5	25.0
1:4:8	.78	12.5	25.0

The measuring of the ingredients of concrete is not very satisfactorily done on the average job. This is because of uncertainty as to the capacity of a wheelbarrow, the

commonly used means of carrying the sand and stone t the mixer. An average wheelbarrow contains from 2 t 3½ cu. ft., depending on the amount it is heaped. On method of proceedure is to make a box just one foot eacl way, inside dimensions, and fill this twice with the bro ken stone and dump it into the wheelbarrow. By observ ing the amount it is heaped each wheelbarrow load can b heaped (or struck off) to the same extent. The same i done with the sand, so that the appearance of two cubi feet of sand may also be noted. Then two bags of ce ment are called one part, or two cubic feet, and eacl wheelbarrow of sand or stone is called one part. Fo hand mixing of a 1:3:6 mixture there would be used : bags of cement, 3 wheelbarrows of sand and 6 of ston or gravel. Four to six men with shovels would be neede to turn this over. A half yard batch mixer could take i the same quantities.

This way of measuring in wheelbarrows is not very ac curate and not very satisfactory on any but mass work, un less the loading of the wheelbarrows is carefully watchec If there is an incline from the stone and sand piles to th mixer, the men are less apt to overload their wheelba rows, as two cubic feet is about all they will care to pu up the grade.

A more satisfactory and more accurate method of me uring the ingredients consists in the use of a bottoml box having handles projecting from the ends. Sucl box made of the proper size to contain a unit quan for a batch of concrete, can be laid on the mixing b and filled level full of sand once and of stone twice two boxes could be used where the volume of stone i double that of sand.) This serves to measure the m ials accurately, but it requires more handling to d them, after this measuring, in an advantageous p for mixing. This adds to the expense of mixing th crete.

One method of using the bottomless boxes me *the last paragraph* is to measure the sand first

a box and then spread it over the board and mix in the cement dry. After this mixing the surface is leveled off and the box placed upon it and filled with the stone. Then the stone is spread over the mixed sand and cement to cover it and the water thrown on and the final mixing done. The sides of the boxes should not be high. A wheelbarrow could not be dumped over the sides of a high box, and a flat box would not necessitate as much spreading of the materials after the box is lifted as a high box.

The bottomless box method of measuring the materials cannot be used in this same way in batch mechanical mixers; a dumping square box, however, could be rigged up from which to load wheelbarrows, and exact measurement could be effected in this way.

There are other ways in which the problem of measuring the materials is solved. The chief points of the problem are to effect the maximum accuracy of measurement with the minimum amount of handling. It is desirable to avoid high lifts of the sand and stone, either in the shovel or in the wheelbarrow. It is also desirable to accomplish these preliminaries to the mixing in the shortest time possible.

The measuring of the cement is fortunately greatly simplified on account of the way in which standard cements are packed. A barrel of Portland cement of the standard size weighs 375 lbs. and contains, when well compacted, about 3.8 cu. ft. When put in a measure and well shaken, this quantity will occupy just about 4 cu. ft. Hence a standard barrel can be taken as four cubic feet. Cement usually comes in bags, four of these being a barrel of cement. A bag of cement can then be taken as a cubic foot. It is important, however, to measure and weigh some sample bags of the cement used on any work to see that they contain a cubic foot and weigh close to 94 lbs. each. If the bags are found not to contain a full cubic foot, the other measures should be made on the basis of the actual volume of cement in a bag.

A bag of cement will sometimes contain lumps. The should be broken up, or, if they are too refractory, th should be thrown out and good cement used to make the deficiency.

A pile of stone in cold weather may sometimes conta lumps of ice. These should be watched for and elimina ed.

It is very important to have an inspector to watch tl mixing of concrete. He should count the bags of c ment; keep tab on the amounts of materials; see that tl mixer runs long enough; see that the consistency aı fluidity is right; see that no blocks of wood, lumps mud or clay, paper from sacks, ice, lumps of hard c ment, etc. go through; see that lumps of hard cement a compensated for with good cement; see that the concre is well mixed, that it is uniform, that it has not balled the mixer, etc.; see that the mixer is clean before star ing; see that the materials are running uniform, regular size and not mixed with dirt, no single wheelbarrows cor posed entirely of large stones or of small stones, etc.

Materials should be moved by power wherever pract cable. They should be stored high where possible. raised to bins when delivered on the job, they may drawn out with use of little labor. This storing of tl materials may often be done by use of cranes. It better to handle the materials in this way than to dun them on the ground and shovel them up. They will cleaner, and the stone is less liable to run irregular.

One method of raising materials to the mixer is to u an incline with a gravity dumping car to material pi This is drawn up to the mixer by cable from the mix engine or other power and allowed to return by its ov weight.

Steel for Reinforced Concrete.

Steel for reinforced concrete should preferably be open hearth steel, though Bessemer steel may safely be used for rods and for plates and shapes that are punched if the punched holes are reamed.

The ultimate strength of the steel is not a matter of much importance, neither is the elastic limit, except as these properties indicate uniformity in the product. It should be a good grade of soft steel. It is of more importance that it stand the bend test of soft steel than that its ultimate strength and elastic limit be high. The reason why high elastic limit and high ultimate strength are not essential characteristics of steel embedded in concrete is the simple fact that these qualities cannot be made use of in proper design of reinforced concrete. This is directly contrary to a great amount of trade literature and some technical literature. Commercial soft steel is almost universally of an ultimate tensile strength of from 50,000 to 60,000 lbs. per sq. in. and a strength at elastic limit of 30,000 to 40,000 lbs. per sq. in. This latter is about three times the safe value that ought to be allowed on the steel, because above this value cracks begin to appear, and there can be no justification for a design that anticipates cracked beams and slabs. There is therefore ample margin of safety in any good soft steel.

If high steels showed smaller elongations for a given unit stress (for stress within the elastic limit) than soft steels, there would be some justification for their use, as they would then not stretch out as much under a given stress, and the concrete would be less liable to be cracked. But the modulus of elasticity is one property that is practically constant for all grades of steel. Even soft wrought iron has a modulus of elasticity almost as great as the hardest steel. The simplest conception of the modulus of elasticity, designated as E, is a unit stress that would stretch a piece of steel out to double its original length, at the rate *at which it* stretches within the elastic limit.

The modulus of elasticity of steel is about 30,000,000, if then a piece of steel is stressed to 10,000 lbs. per sq. in., it will stretch one-three-thousandth of its length. Beyond the elastic limit different grades of steel exhibit different characteristics. Soft steels stretch out more before failure, while high steels and soft steels that have been rolled or drawn cold or twisted cold, break without much stretch or reduction of area at point of fracture. This lack of stretch beyond the elastic limit is held out as a benefit in trade literature. It is a positive detriment. If failure occurs in steel that will not stretch, it will be sudden and without warning, whereas if the steel stretches out, it will allow a beam or slab to sag before failure. Besides giving warning of failure the sagging will in many cases reduce the stress in the steel very materially. The author has seen tests of slabs reinforced with soft steel that sagged enormously and could not be broken.

The reason why it is important that steel stand the cold bend test is because rods are very often curved and bent in construction. This bending should be done cold, for if the steel is heated, its internal structure is changed, and annealing would be necessary to restore it. Soft steel of ordinary manufacture will, in general, stand more punishment than harder grades of steel. The threading of rods and punching of plates or shapes are less liable to cause incipient cracks or hardened metal in soft steel than in high steel. These are also processes to which the embedded steel may be subjected.

Special steel, while it has a high sound, does not possess any needful characteristics, as an element in reinforced concrete, that are not possessed by the cheap commercial article. This is true because of the limitations of the concrete. Good soft steel is not a special steel but is the commonest product of the steel furnace. It is important, however, that it be good and that it be soft, that is, not a high carbon steel.

There should be a wide margin of safety in the amount *that a steel rod* will bend. A piece of steel of high ulti-

mate strength may stand a bend of 100 degrees and fail if bent 105 degrees. It is clear that this steel would not be fit to use where it is bent at an angle anywhere near approaching this.

The best carbon steel for structural purposes is found to possess an ultimate strength between 55,000 and 65,000 lbs. per sq. in. Formerly two grades of steel were recognized in most specifications for structural steel work with allowed limits that overlapped two or three thousand pounds around the 60,000 mark. The result was that manufacturers usually succeeded in making nearly all of their structural steel within the overlap, so that it would fill either specification, though a wide difference in allowed unit stresses was sometimes permitted. The allowed limits given at the beginning of this paragraph represent a steel that is a mean between the older grades of soft and medium steel. It is also about midway in tensile strength between the very soft steel now used for rivets and the higher steel used for eyebars and other forgings. It has been found by experience to be satisfactory for structural purposes. For reinforced concrete it satisfies every structural requirement, and because of its availability it is the most economical material that can be used.

Tests made to see that the ultimate strength lies between hese limits are very useful to ascertain that the steel is :gular in quality. Dead soft steels may be ruined in the anufacture. High steels will not stand bending.

The elastic limit of the steel should be not less than one-lf of the ultimate strength and the stretch in a measured gth of 8 inches should be not less than 24 per cent. It uld stand the cold and quench bend test, 180 degrees without fracture. In the quench bend test steel is ed to a cherry red, as seen in the dark, and quenched ater at ordinary temperature, before bending.

ie proper use of steel in concrete is in small sections distributed throughout the mass. The bars should be ated from each other to give the gripping effect of oncrete full play. If they are placed in a layer close

together, a cleavage joint is formed and the concrete is liable to break off. This would be the case in a close coil or set of rings close together or in a set of rods lying close together near the bottom of a beam. Plates or sheets of steel should not be used as separators or spacers for rods, as these will also form cleavage planes. If heavy rods are used, surrounded by comparatively little concrete, the concrete is unable to grip the steel and the differential expansion due to change in temperature will crack the concrete. Pyramids of concrete surrounding the bases of out-door steel columns seldom last through a winter without being cracked. Girders covered with a shell of concrete would be subject to the same detrimental effect of differential expansion.

The steel should not be placed close to the surface of the concrete. It cannot be gripped properly unless it is deep enough in the concrete for the latter to take hold. It cannot be protected from rust and fire unless there is some concrete between the steel and these destroying elements. It is bad practice to lay the steel on the forms and then the concrete on this. The steel is neither properly protected nor gripped by such means. The depth to which steel should be buried depends upon the size of the section. It is manifest that the heavier the section the more concrete is needed to grip it and to overcome differential expansion. If rods are bedded deeper they will be less affected by external change of temperature. Heavy rods are of more importance in a structure, hence their protection is of more vital importance than that of light rods.

Standard sizes of rods or shapes should be used as much as possible, so that they can be obtained without delay from the mills. Also as few different sizes as possible should be employed. Simple details are essential. A complex structure will be difficult to surround properly with concrete. There should be no broad flat surfaces to work the concrete under.

The steel work should be designed with a view of its being easily placed in proper position and held there

ıst the displacing tendencies due to the placing of the rete. Where rods cross, they should be wired to- er, and this should be done before the forms are ed to the point that they will interfere by preventing access to the rods. Extra wires may often be used dvantage to tie the rods in place. These wires may e the further purpose of holding the sides of the forms ı spreading. They can be cut off at the surface when forms are removed.

ods in the bottom of a slab can be kept from lying on bottom of the form by placing small stones under them before the concrete is placed.

here is nothing better than round rods for reinforced rete for the reason that positive end anchorages can nade by means of nuts and washers, or large square es having threaded holes, and because splices can be e by means of sleeve nuts. Both the anchorage and splice will take nearly as much stress as the full sec- of the rod. No other kind of anchorage or splice has same efficiency. The thread and nut detail comes into immediately without any slip. Rods laid together bound would have to slip before receiving much stress hooks and curves are no doubt of some aid as a autionary measure; they may show strength after [illegible] straightening of the rod, before ultimate failure [illegible] k cannot be seriously considered as an efficient end an- age for a rod taking practically its full strength near end.

Vire cables in concrete are faulty from every [illegible] sidered. The stretch in a wire cable [illegible] ı a solid steel rod, as shown by tests made at [illegible] n Arsenal in 1896 may be as much as four times [illegible] he plain steel rod under the same unit stress. The unit allowed on wire cables [illegible] ıbout four times as much as that on plain steel [illegible] stretch in the steel cable may be [illegible] n the plain rod when [illegible] ed. The placing of [illegible]

scarcely do more than take out the curving tendency due to the winding of the coil. A stress of any magnitude on the cable could scarcely be resisted by the supporting walls or beams.

Sharp bends should not be made in steel rods under stress, as the concrete cannot resist the stress at the knuckle. Curves with a radius 20 times the diameter of rod are permissible.

No welds should be allowed where the steel is under stress approaching that allowed on the full section. If rods were found to be a little too short to reach between supports, it would not be harmful to weld a short piece on. In general all welds in steel should be avoided.

Steel that is to be placed in concrete should be free from scale and thick rust. A thin coat of rust is not objectionable. It is well to let new rods rust so as to loosen the mill scale. The scale should then be scraped off or brushed off with wire brushes. The coating of steel with grout to preserve it from rust is a doubtful expedient. The thin layer of cement dries out and does not set properly. It may not bond with the cement of the concrete. It is better to have the fresh concrete come in contact with the steel. A more intimate union is effected.

No paint should be used on any steel embedded in concrete. Grease and dirt are objectionable and should be removed before the steel is placed.

Steel should be placed well in advance of the concrete, so that any delay in placing it will not hold up the placing of the concrete.

Vertical reinforcement in hooped columns should be of smooth rods so that when the concrete shrinks in setting it will not be prevented from settling down.

No reliance should be placed on a short length of a rod either plain or deformed to act as anchorage. Where two thin walls intersect they should be reinforced with steel. If there is no special stress acting on the walls it is enough to curve or bend rods around the corner. If, *however, there* is pressure on the walls, as in the case of

the walls of a rectangular cistern or the analogous case of the bottom slab and rib or counterfort in a retaining wall, the bend in the rods would be too sharp to be effective. In such case a short length even of a deformed rod is entirely inadequate as an anchorage. The best form of construction is probably a steel angle in the corner punched to receive rods with nuts on the ends.

Steel rods are sometimes laid in the ground, as in the case of those used to tie the shoes of steel arches together in such structures as train sheds. To protect these from corrosion a good plan is to wrap them in canvas soaked in Portland cement grout and then to paint the canvas thickly with grout.

Handling and Placing Concrete.

After concrete is taken from the mixer or the mixing platform it should be placed as soon as possible in the forms. As a rule the sooner it is placed the better. There is, however, a possible exception to this rule. It is found that concrete that is placed under water is less liable to have the cement washed out of it, if it be allowed to stand even for a period of two hours or more before being placed. It should, however, be mixed again before being deposited. Concrete placed under water would in general be in a thick mass, so that any weakening due to partial setting before being placed would not be of so much consequence as it would be in such work as reinforced concrete.

It has been found also that retempered concrete will adhere better to concrete that is set or partially set than freshly mixed concrete.

Generally concrete should be placed in less than ½ hour after mixing. It should not be disturbed more than absolutely necessary after being put in place. If a receptacle is used to hold the mixed concrete temporarily, the concrete should be taken out of it in the order in which it is put in. If *delay occurs* in the placing of the concrete

that results in leaving a batch mixed but not placed before the end of half an hour, generally the mixed concrete should be rejected. However, if it is thoroughly re-mixed, adding a little water, if necessary, it may be made quite good for the purpose. It is found that retempered concrete is almost as good as freshly mixed concrete, if the second mixing is well done. For walls and heavy work, re-tempered concrete would not be objectionable. In columns and floors the uniformity should not be disturbed by using an occasional batch of concrete that has received a different treatment from the rest of the work.

Jarring of the concrete in wheeling from the mixer to the forms should be avoided. This jarring will tend to separate the stone from the mortar. The runway for the wheeling should be smooth. The concrete should not be dropped far from the mixer, to the wheelbarrows or carts, as this also separates the ingredients.

The mixer should be placed high and the ingredients raised to it, so that the concrete will not have to be lifted after it is discharged from the mixer. The concrete should be handled as little as possible after leaving the mixer.

After the concrete is placed it should be left undisturbed until it has received a hard set. To this end care must be used in placing concrete beside other concrete that is partially set. Jarring the projecting steel rods with the buckets or carts in placing fresh concrete will impair the bond of work that is partially set. Wheeling carts over newly laid floors will have the same result. The work should be planned so that concrete farthest from the mixer will be deposited first so as to avoid this.

Jarring the forms by the buckets or carts in which the concrete is handled must also be avoided. In putting in concrete piles the driving of the core for a pile near one that has lately been placed may disturb the latter. Walking over newly laid concrete should be avoided as much as possible.

The amount of concrete that should be placed at once ends upon the kind of construction and the kind of

concrete. Comparatively dry mixtures, which, in general, should only be used in massive, unreinforced work, should be placed in layers of about 6 inches of thickness and rammed. In such work concrete may be carried in half cubic yard or cubic yard buckets, handled with a derrick.

In reinforced concrete, where semi-liquid concretes must be employed, it is important that the concrete be poured in such way and in such amounts as not to cause air pockets to form. The same is true of any concrete that is to be impermeable. Ramming is not essential in such concretes. In fact, if the concrete is of such consistency that it can be rammed, it is unsuitable for the purpose. In narrow forms only small quantities should be dumped at a time. The liquid concrete should be stirred and worked around so as to make it flow into the corners and around the reinforcing steel.

If large quantities of concrete are placed at a time or in one place in reinforced concrete work, it may cause springing of the forms, or it may bend or displace the reinforcing rods or any rods that are used to brace the forms.

Columns more than about twelve feet high should be poured from a point half way up, so that the concrete will not have so far to drop and so that puddling of the concrete around the reinforcement can better be effected. Then the doors left for this purpose can be closed and the remainder of the column poured. Special care is needed in puddling the concrete in columns to make it flow around the reinforcing rods.

Large quantities and thick layers of wet concrete may be placed at once in mass work, if it can be done without leaving air pockets. The reason for comparatively thin layers in rammed work is so that the ramming will be effective.

Cinder concrete should be tamped lightly if at all, as heavy tamping or ramming will break the cinders. A wet *mixture is most suitable* for concrete made with cinders. *It is often necessary in* exposed work to spade the con-

crete against the forms so as to work back the large stones and to bring the mortar to the surface. If thick layers are deposited at once, this cannot well be done, and the appearance will suffer.

If one or more batches of concrete are too wet, as exhibited by free water on the concrete after being placed, a comparatively dry batch or more should be mixed to take up the surplus water.

Rammed concrete should be placed in layers approximately horizontal. If this kind of concrete is used in arches the layers ought to be normal to the line of thrust. This is difficult to do without setting up temporary forms normal to the arch ring and tamping beside these. This is an argument for the use of wet concrete in arches. With wet concrete it is not important to have the layers normal to the line of thrust except at quitting time. Where it is possible, the work should be planned so that the entire ring of an arch can be poured without intermission. Another satisfactory plan is to lay successive rings ending each against a temporary vertical partition. This could be removed in a day or two, when the concrete would still be green enough for the next ring to adhere pretty well Another plan is to pour half of the arch ending it with a vertical surface at the crown.

In exposed walls the layers should be kept higher on the face especially where there is any tamping. Tamping brings the cement to the surface and this makes a relatively impervious layer. If this layer slopes out toward the face, any water in the concrete will be shed outward, carrying with it any dissolved salts to the face of the wall. The evaporation of this water leaves the salts as an efflorescence on the wall.

It is important that concreting be stopped, when discontinued, at a joint where the strength is not impaired. To effect this the finishing surface, as stated, should be normal to the line of thrust and it should not be where there is any *considerable shear.* In a column the surface should be *horizontal and below* the line where beams join in. It is

preferable to pour the columns some hours before the beams and girders so as to allow some time for settlement and shrinkage. It is better to pour an entire floor at once, but if stops must be made, they are best made at the middle of the span of a beam or slab the latter being on a line parallel with the beams; both should be against vertical surfaces. A beam should not stop off near the support, as the shear is great at such section. Where a beam and slab are figured as a T beam, both should be poured at once. Where the beam is figured as rectangular, it is not so important that both be placed at once; however, it is better, as the full depth of beam includes part of the slab. A notch could be left for the support of the slab, and thus nearly all of the beam would be placed at one time.

Some engineers advocate splitting a beam or girder in two in the middle, longitudinally, by a temporary vertical bulkhead, when it is necessary to stop in the neighborhood of a beam or girder. The author would not recommend this, but would rather leave a step the depth of the slab, an inch or two wide, at the top of the beam. The slab would find support on this step, and if the beam is figured as a rectangular beam as it should be and not as a T beam, it can borrow from the other slab for what may be lacking in bond at the step. In a properly designed beam carrying slabs there will, of necessity, be an excess of compression area. It is better to rely upon this than to go to the trouble and expense of fitting a bulkhead along the center plane of a beam, with all of the added difficulties this entails, such as narrowing up the space to work in, possible displacing of the steel, different degrees of shrinkage contraction in the beam, etc.

In enclosing steel work concreting should not be stopped at or near a broad horizontal surface of steel, as the concrete will shrink away from the steel, and the thin crack cannot be filled up. It is best to stop several inches below or above such surfaces.

In walls, where possible, the stop should be made at a *horizontal or vertical* bead or groove, so that the line

at the junction of two days work will not show. There are several advantages in placing walls in short lengths at a time. Vertical joints do not show up so bad as horizontal ones, especially when these are made by use of a bulkhead and are true to line. The contraction of the concrete due to setting will be equalized as the wall progresses. These vertical joints will often be sufficient to take up expansion and contraction due to change in temperature. The mixing plant can be moved from section to section, thus lessening the distance through which mixed concrete must be carried.

Before starting to place concrete it is important to see that the forms are clean. The bottoms of boxes for beams and girders and the bottoms of columns should be cleared of all dirt, sawdust, shavings, blocks of wood, etc. Blocks of wood may have lodged among the reinforcing rods of walls or columns. These must be removed.

It is important to have the forms finished far enough in advance of the placing of the concrete to insure continuity of the concreting.

When leaving off the placing of concrete for the day care should be taken to see that the finishing surface is left so that the conditions of continuous work are approximated as near as possible. The bonding of the next days' work should be made as good as the conditions will permit. It is better, in general, to leave the surface rough than smooth. In heavy work a bond may be made in several ways. One way is to make steps in the top by setting up vertical boards and tamping against them. Another way is to bed large stones half in the last layer of concrete allowing the other half to project into the new concrete. Still another method is to lay wooden blocks in the concrete to form recesses and remove them before the next day's work begins.

In reinforced concrete work and in impermeable concrete dependence must be placed more upon treatment of the surface before starting to lay concrete than on roughing it at quitting time. It is important in any work that

le surface be clean. Also the forms should be cleaned of ny concrete that has been spattered on the day before. 'icking over the surface with picks, if it has stood long, , recommended to remove the top skin. Washing the ırface with a hose and brooming or brushing it would e sufficient, where it has not stood very long. This washıg of the surface of the last day's placing of concrete is ery useful to get rid of the laitance or slime that comes) the surface. It serves to lessen danger of efflorescence nd to make a better bond. The bond will be strengthened y brushing over a thin coat of grout of neat cement or by ifting neat cement on the previously moistened surface. ;ond to old and hardened concrete is effected by this same ıeans.

Much cement pavement work is being done in Pittsburg y picking over the worn-out flag stone pavement to two nches or so below the desired new level, moistening the urface, dusting on neat cement and spreading the same vith a broom, and then laying cement mortar to the deired level.

When concrete is placed in contact with bricks or porous ile, these should be thoroughly saturated so as to imrove the bond and to prevent the absorption, by the porus substances, of the water in the concrete.

A very good grade of concrete blocks or artificial stone an be made by casting the blocks in sand as iron castngs are made. The mixture must be very wet, a consistncy that would be called soupy. The surplus water is bsorbed by the sand, and this serves to keep the block noist during setting. By using a well selected aggregate, uch as crushed marble, crushed granite, white sand, etc., xcellent artificial stone can be made. The product is lense and uniform, because the concrete is not dry and amped, and because varying atmospheric conditions have ıo effect upon it while it is setting. These artificial stone locks are usually cast smooth and then tooled on the surıce by tools operated by power. A pleasing surface is ıde by the use of carborundum wheels. If reinforced

with steel, the blocks may be made quite thin, even down to 1½ or 2 ins. Blocks not reinforced, used for veneering, are usually 3 or 4 in. thick. The blocks should remain in the sand 4 to 6 days, and should then season for about two weeks before being used.

Fine details in ornamental work will be more satisfactory if cast in sand with very wet mixtures than in many other kinds of molds or with dry mixtures.

Concrete blocks can be made of a medium mixture, molded under heavy pressure. Because of the pressure they can be removed from the mold immediately. The blocks are denser and better in every way than the hand tamped blocks made from a dry mixture.

Large concrete blocks are often cast near the point where they are to be placed in a structure in suitable forms of wood or sheet steel or other material. When hardened, they are lifted into place. Very heavy blocks can have rings cast in them to facilitate handling. These blocks may be used for breakwater construction, sea walls, arches, etc. In sewer construction curved blocks can be cast and the necessity of expensive arch forms and lagging avoided. The arch blocks can be held in place by the side blocks, and little or no centering is required. In like manner reinforced concrete slabs can be cast separately for the top of a sewer, and as these exert no thrust the side walls can be thinner.

The casting of reinforced concrete slabs may often be found to effect a saving in the construction of sidewalks and floors. These can be cast one over the other with some kind of a separating medium and lifted into place when hardened.

Beams and columns have likewise been cast on the ground and lifted to place. In general this is a doubtful expedient. The importance of the beams and columns of a structure being tied together as a unit is very great, unless the walls of the structure supply all of the lateral rigidity.

Concrete piles are made both by casting them in place or casting them on the ground and then driving. In the first method the hole for the pile may be made by driving a wooden pile and withdrawing it; or it may be made by driving a collapsible core with a sheet metal shell and filling this shell with concrete; or it may be made by driving a steel tube with a removable driving point and filling the hole with concrete as this tube is drawn up. In any of these methods the concrete should, in general, be well rammed so as to insure filling of the hole and the ability of the pile to take a load without settlement. Piles that are cast before being driven should be cast upright, if made of dry concrete, so that the joint between the layers will be normal to the direction of the driving. In wet concrete it is not important whether the piles are cast horizontal or vertical. However, unless they are well reinforced it is difficult to raise long piles to the vertical position. As far as practicable, the length of these piles should be known before they are driven, as it is not practicable to splice them, and it is difficult and expensive to cut them. In driving concrete piles a hammer weighing nearly as much as the pile and having a short fall is needed. The blow of a light hammer will be absorbed locally and shatter the pile. Concrete piles should stand two weeks or more of good weather before being driven. Concrete piles should be larger in diameter and generally fewer in number than wooden piles for the same structure.

In placing concrete under water it is important that it be treated in such way that the cement of the concrete will not be washed out. Dropping concrete through a depth of water will not only wash out some of the cement but will tend to separate the ingredients. Concrete should not be rammed under water, as the stirring of the water will carry away cement. It should not be deposited in running water.

If concrete is to be placed in water, it should be placed *in as large batches as possible*. It should be wet concrete, *so as to require no ramming*. It should be mixed long.

Concrete that is mixed and allowed to stand several hours and then mixed again is said to be preferable to freshly mixed concrete for placing under water, as the cement is partially set and is less liable to be washed out. It is well to use about ten per cent. extra cement for the concrete that is to be placed under water to allow for loss.

The concrete may be lowered in steel buckets with bottom doors that are opened when the bucket reaches the bottom. Canvas sacks may also be employed. These sacks are lowered with the mouth down. This is tied shut in such way that it may be tripped open with a line. A better way to place the concrete is to let it down through a tube or tremie. This should be kept full of concrete, the lower end resting on the bottom and being moved about so as to distribute the concrete.

Reinforced concrete should, in general, not be placed under water. Any concrete reinforced with steel that will be submerged during setting and subsequently should be designed so that no dependence will be placed upon the adhesion or grip exerted between the concrete and steel. Concrete setting under water does not shrink and grip the steel as that which sets in air. The rods should have nuts on the ends and washer plates or some other effective end anchorage. A riveted structure under water embedded in concrete for its protective value, is legitimate construction, if the concrete is of sufficient mass not to be cracked by differential expansion.

When grout is to be used under water, as in filling interstices in stone work, cracks in concrete, etc., neat cement should be employed, as sand will separate from a mixture of sand and cement in passing through water.

Concrete should not be deposited in polluted water, as that containing sewage or discharge from pulp mills or refuse from other washing processes. Such water coming in contact with fresh concrete will destroy it by attacking the setting cement.

In constructing inverts, such as the curved bottoms of filter beds, sewers, etc., it is often best to omit the lag-

ging as far up the side of the curve as the concrete will permit without sloughing. The arch forms, however should be in place to serve as guides in finishing the surface. By omitting the lagging the concrete can be compacted better, and a better surface finish can be obtained.

Much time and labor could be saved in the making of concrete if the broken stone or gravel could be placed without having to pass it through the mixer or having to pick it all up in shovels and turn it a half a dozen times or more. Such a process would not be productive of the best grade of concrete because of the many chances of air pockets being left and of lack of bond with the stone due to failure of the grout to flow beneath the stones. There are some situations, however, in which the kind of concrete resulting from this process would meet all of the requirements. This method of laying concrete for street pavements is described in Concrete Engineering, Apr. 15, 1907, in a paper written by Mr. Walter E. Hassam. The method there described is as follows. First the subgrade is rolled to an elevation, for ordinary street paving in concrete, about 6 inches below the finished surface. Then broken stone of the egg size is spread to a sufficient depth so that after rolling it will be 2 inches below the finished grade of the street. The following is quoted from Mr. Hassam's paper.

"This foundation stone is rolled or compressed until thoroughly compact, and the voids reduced to a minimum. It is then treated with a grout, composed of one part of cement to 4 of sand. This grouting and rolling is continued, until all the voids are completely filled. This process gives an exceedingly dense concrete, which is very strong.

"For the wearing surface, there is spread upon the foundation, before it has set, sufficient stone, of the stove size, to bring the street to the required grade after rolling. This stone is uniformly rolled or compressed, until embedded in and united with the foundation. Then it

is given a thin grouting of Portland cement and sand, mixed in the proportion of 1 cement to 2 sand.

"The voids are thoroughly filled with grouting, and then the surface is rolled until the grout flushes to the top of the stone. As a finish, there is then applied a thin layer of creamy cement and pea stone mixed in the proportion of 1 cement, 1 sand and 1 of pea stone.

"This surface is poured on, brushed and rolled to an even surface. The street is then allowed to set for at least 6 days, when it is ready for traffic. The layers follow each other so closely that the foundation does not set until the whole is complete. When complete, the entire road is a solid, homogeneous mass of rock and cement, that will resist anything that can possibly come in contact with it.

"The finished surface of the pavement presents to the casual observer a smooth and fine appearance, but, on close examination, it is found to be somewhat rough, so there will be no slipping of horses or skidding of automobiles."

The above is the method of laying concrete used with success in Worcester, Mass. It may be used for the concrete foundation of a brick or block pavement of any kind or alone for solid concrete pavement. In a note regarding these same pavements, in the Engineering Record, Vol. 53, p. 625, it is stated that for a cement wearing surface a thick grout of sand and cement is poured over the foundation (of rolled stone) and immediately filled with fine crushed stone and rolled.

It is certain that concrete made as above described, with thorougly mixed grout, would be equal to if not superior to the half mixed commercial concrete that often goes into our city streets. The rolling and consequent compacting of the broken stone before applying the grout produces a foundation that is capable of supporting considerable load without the aid of the cementing grout.

Other cases where concrete may be made in place, with*out handling the broken* stone in the mixer are in rough

retaining walls or breakwaters. In these the grout may be introduced by inserting steel pipes at intervals and forcing the liquid mortar into the voids, either by gravity or by air pressure.

The Setting and Hardening of Concrete.

It is important that concrete be free from jar or disturbance during setting. It should not be subject to intense cold or high heat. Water in small interstices, as in the body of concrete, will not freeze at 32 degrees; but it will freeze at a somewhat lower temperature. If the materials can be kept above the freezing temperature until the concrete is placed, danger of freezing is lessened by the heat evolved in the cement during the process of initial set; so that temperature higher than about 25 degrees can be worked in without much danger. This is especially true of concrete in thick masses. In thin walls or slabs the heat generated will be quickly lost and protection is needed.

Protection of setting concrete may be afforded by the use of tar paper or canvas or boards laid over it. A foot or so of hay is good for this purpose. Two layers of canvas or tar paper, separated by boards, will give very material protection. If only a single layer is used, it should not be allowed to touch the concrete, but should be kept out by boards. If manure is used, it should not be allowed to come in contact with the concrete, and it should not be allowed to become wet. Water that has absorbed elements from the manure is apt to be injurious to the setting concrete and to cause it to rot and be useless. Cement bags or tar paper used for protection should be well lapped.

Placing of concrete in temperatures below 25 degrees should be avoided where possible. If it must be done, the best thing to do is to enclose the work with canvas and heat the enclosed space with salamanders, or better with steam. The concrete should be protected from the direct

heat of any kind of stoves, so that it will not be dried out too soon and prevented from setting.

Though concrete that has been allowed to freeze and afterwards to thaw has, after having had time enough to set, taken on apparently the strength of properly treated concrete, it is not safe to rely on concrete thus treated in any structure where strength is an essential feature. Freezing should be avoided and prevented by means that will not heat up the concrete and cause drying out. The use of salt in the water is not recommended. Anything short of a strong brine would freeze at a temperature not much below 32 degrees, and the possibilities of efflorescence and of corrosion of embedded steel make the use of salt an undesirable risk.

While thoroughly seasoned limestone concrete may stand 400 to 500 degrees F. without any detriment or change of structure, and other concretes may stand more, it is not safe for it to be subject, while setting and hardening, to a heat that will evaporate the contained water. The presence of water is necessary to the hardening of the cement, and, if it be robbed of this water, it will suffer in strength.

Concrete that is setting will suffer from other than thermal conditions which would not effect seasoned concrete. Water containing decaying organic matter, sewage, the discharge from pulp mills, etc. will rot setting concrete, though these substances will not, in general, have a deleterious effect on hardened concrete. Some oils will weaken setting concrete which could be safely stored in concrete tanks.

Water is necessary to the hardening of cement. The water of mixing, if it be a liberal quantity, is sufficient for this purpose in some cases, as when the concrete is in a damp place; but it is generally best, and sometimes necessary to the safety of the structure or the integrity of the concrete, to add water during setting. Concrete should be *covered and protected* from the rays of the sun and from *wind to prevent* evaporation of the water. Concrete *blocks (the kind* that are made of a concrete having the

consistency of "moist earth" and tamped in molds, and from which the molds are immediately removed) are greatly improved and made to approach the condition of good concrete, if they are immediately soaked, upon removal from the forms, by allowing a smooth stream of water under no pressure to run upon them until they will take in no more. Generally these blocks are kept sprinkled with a little water for several days to "cure" them. It would be better to use preventive measures and forestall the ailment by mixing them with plenty of water and allowing the molds to remain until the concrete will stand up. Not much can be expected of a block so porous as to turn dark gray after a rain, even if cellular construction does keep the inside of the wall comparatively dry. Disintegration is almost sure to get in its work. No natural stones that are not compact would be acceptable for building work.

Rich mixtures of concrete need especially to be kept moist during setting, as these are more apt to shrink and crack on the surface or in the body of the concrete. A rich mortar finish or a troweled surface should be kept wet for nearly a week and protected from winds and sun to insure its solidity.

Any thin coating of mortar or grout should not only be put on a thoroughly saturated surface, but should be liberally wetted for a day or two. Such coatings are apt to have their water absorbed by the wall or evaporated and to lose their cohesion.

When a concrete wall or pier is placed in a cofferdam, it is well to let in the water around it a day after the concrete is placed. Concrete requires longer to set under water than in the air, but it acquires greater strength. Specimens that have hardened in water will show much greater strength than those that have hardened in the air. Immersion in water should be delayed until initial set *takes place. Moistening* concrete will delay the setting to *some extent. This should* be taken into account in gag-

ing the time to remove the molds. Humidity in the atmosphere acts in some degree like immersion in water.

The time that should elapse between the placing of concrete and the removal of the forms depends upon a number of things, among which are the consistency of the concrete, the richness of the mixture, the load sustained, and the temperature and atmospheric humidity. Wet concretes require longer to harden than dry concretes. Lean concretes require longer than rich ones. Concrete hardens more slowly under water or in a saturated atmosphere than in dry air. Low temperatures delay the setting of concrete. If the temperature be below freezing, the setting may be suspended. Failures have resulted on account of forms being removed from concrete that was frozen and appeared to be hardened due to setting.

Another error apt to be made is to mistake drying for setting. Drying is not a necessary accompaniment to the hardening of concrete. In fact if the concrete is too warm and the air too dry the early drying of the concrete that will result will be detrimental to its strength. Concrete should not be allowed to dry out until it has stood for several days. Sidewalks should be sprinkled for four or five days. They should be covered and protected from currents of air. Plastered work and reinforced concrete need special care in the matter of maintaining moisture on the surface; otherwise shrinkage cracks will develop.

Concrete blocks need frequent and copious sprinkling, which should be continued for a week or more.

Concrete receives its set when it reaches the state where a change of shape cannot be produced without rupture. This requires from a few minutes, in rich mortars of quick setting cement, to several hours, in lean mixtures. A common way of determining when concrete has set is by pressure of the thumb nail. After the set has taken place the concrete continues to harden and gain strength for months and sometimes for years. In ordinary weather nearly the full strength is attained in six or eight weeks. Loading tests may be made at this stage. Strength necessary to

support its own weight is reached at varying periods depending upon several conditions.

In counting the time that concrete should stand before removing the forms days when the temperature is at or below freezing should be counted out, or at least allowance should be made for almost total suspension of the hardening process.

It is safe to remove the forms from mass work, receiving at the time no load except its own weight, in from one to three days; in warm weather with dry concrete, one day, in cold or wet weather or with wet concrete, more time. When the concrete will bear the pressure of the thumb nail without indentation, it is ready to support itself in this class of work. Thin walls should stand two to five days. Slabs of reinforced concrete should stand about one to two weeks of good weather before being called upon to support their own weight. Slabs of long span may require more time than two weeks. At the same time that the slab centering is removed, or even before it is taken down, the forms on the sides of beams and girders can be removed, leaving the supports of the bottoms in place for a longer time. This will afford an opportunity to inspect the surface of the beams and girders and to plaster up any cavities before the concrete is too hard. Where practicable it is well to leave the shores under beams and girders for three or four weeks. Large and heavy beams should be allowed to stand longer than short ones, because the dead weight is a greater fraction of the load they are designed to carry.

Column forms can be removed in a week or so, if the entire weight of the beams is supported by shores close to the columns, otherwise three weeks or more should be allowed.

Arches of small span can have the centering removed in one to two weeks. Large arches should harden a month or more. Where practicable it would be well to leave the concreting of the spandrel wall of an arch span until the arch ring has hardened and the forms are removed. The

settling of the arch often cracks the spandrel wall and gives an unsightly appearance to the bridge.

Ornamental work should have the forms removed as soon as possible, so that defects can be plastered up and so that swelling of the wood will have less time to act.

Falsework should be removed carefully, without jar to the concrete either by hammering on the boards or dropping heavy pieces on the floor below. The supports should not be removed when any unusual load is on the floor. Materials should not be stored on floors that are not thoroughly hardened and self supporting.

Concrete in reinforced work should ring when struck with the hammer, before the supports are removed.

Finishing Concrete Surfaces.

One of the chief difficulties in connection with the use of concrete is to get a surface finish that is pleasing in appearance and at the same time economical. The various methods in use will be taken up with a view of showing their good and bad features and their limitations.

Surfaces that are wrought in other materials than concrete will first be considered.

A common and acceptable surface finish is a veneer of brick. Brick work in 4 in. thickness may be laid against a concrete wall. For example, if a 13 in. wall would be required, an 8 in. wall of concrete may be put up in the regular way, using wooden forms, then the brick may be laid outside of this. Metal bond is often used, small pieces of wire or other metal being bedded in the concrete and projecting out to be built into the brick work. An other than metal bond is preferable. The metal is not thoroughly protected in a brick wall because of the porosity of the wall. Occasional belt courses of cut stone projecting out for the support of the brick work would serve to lessen the height of the unsupported brick veneering and thus lessen dependence on metal bond. A good method of bonding would be to leave vertical recesses, at intervals, about

: width of a brick. Into these headers could be laid and ıs a very satisfactory bond secured.

Another way to have a brick surface is to lay up the ick wall and use it as the outside of the form pouring : concrete behind it.

Cut stone and artificial stone veneers may also be used the same way as brick. This is satisfactory for a wall t taking much vertical load. It may lead to structural :akness, if used for a pillar taking a concentrated load. ore than one case could be cited in large buildings, ıere reliance upon a combination of a stone shell and a re of other material in a pillar, to take a load, necessited the removal of the pillars and the insertion of steel lumns after the walls were completed. Cracks in the one veneer showed that it was taking the load, and that was not capable of withstanding it. In one building c core was of rubble masonry and the shell was of cut one; in another the core was of concrete and the shell as of artificial stone. Both rubble masonry and con'ete will shrink on account of the large proportion of ortar. The cut stone and the artificial stone, with their :ep courses and thin mortar joints, do not shrink any :rceptible amounts. The result is that about all of the ad must be carried by the veneer. The same fault has :en observed, where tile facing was backed with brick. ı this case the cracking of the tile was attributed to ıilure of the mortar to set because of the cold weather. ettling of the brick work behind the weak tile would, owever, be apt to produce the same result in work set up ı warm weather. In construction of this sort no dependnce whatever should be placed upon the veneer in suporting the load, and it should be built in such way as to llow the core to shrink in setting. This might be effect:d by using wooden blocks a trifle higher than the stones or one or more courses and removing these later and subtituting the stone or tile. It might also be done by leavg out the top course of stone or tile facing. Or a number the joints in the facing might be raked out, soon after

the concrete backing is placed, and, after the concrete has set and shrunk, these joints could be pointed. The use of dry tamped concrete would lessen the shrinking. In a long wall brought up slowly the shrinking will not be so harmful. In a small pillar a cast iron or steel column should be used in the middle.

In the present state of the art artificial stone or cut stone facing is probably the best surface treatment for concrete in such construction as residences and office buildings. This is partially due to the fact that concrete workers have not developed the skill that workers in the other materials possess. Brick work with the outer ½ in. or so of mortar "raked out" (or blocked out with wooden cleats, as is done in practice) makes an appropriate and pleasing surface finish for rugged styles of architecture.

Blocks in glazed tile are made use of for external finish of buildings. With these it is even more important that but a short height be laid at once, if concrete is poured behind them, or some other precaution be employed to prevent their receiving any of the load of the wall. These tile are brittle and unreliable in supporting loads.

Thin tile can be used as an external finish by pasting them on paper, as they come for pavements, and then pasting the paper, with bill posters' paste, on the inside of the forms before concrete is placed. Before the forms are removed they must be thoroughly flushed with water.

Boulder facing may be made on rustic walls or arch spans by placing the boulders against the forms and then the concrete behind them.

Of the surface finishes that are made in the concrete itself there may be mentioned those that are made in the concrete by aid of the forms, while it is being placed, and those that require subsequent treatment.

If the concrete is simply placed against the rough surface of sawed boards, it will have the impression of the saw marks and grain and knots as well as the cracks or joints. This is far from pleasing for any surface above *the ground.* When the forms against the exposed face are

made of planed boards, tongued and grooved, and neatly jointed, the surface is greatly improved. A broad surface can be relieved of its monotony by paneling. The larger and rougher the surface the bolder the panels should be. Long retaining walls should preferably have panels to relieve the dead flat surface.

If the concrete is thrown indiscriminately against the forms, the surface will not present a smooth appearance, especially if dry concrete is used. By manipulating the concrete with spades or shovels it can be given a richer and smoother surface. As a layer of concrete is placed a shovel is run down against the form and the larger stones shoved back. This allows the mortar of the concrete to flow against the form, and a surface of mortar results. Sometimes a perforated shovel or spade is used for this purpose, and the mortar passes through the perforations. With rammed or dry concrete the spade may be used to shove back the concrete and a wetter mortar poured in. Of course the smoother the forms for this class of work the better will be the appearance of the work.

In narrow forms it is recommended by one engineer that instead of a spade a hoe be used with the blade bent nearly in line with the handle.

Another way by which a mortar finish may be made is to plaster the forms with cement mortar in advance of placing the concrete. The mortar and concrete are united by tamping.

Another way to obtain a mortar finish is to place a loose board against the forms and tamp the concrete behind the board. The board is then removed and mortar run into the space that it occupied. This is of course only applicable to comparatively dry concrete.

Still another method is to use a sheet of steel on the one side of which are riveted 1" x 1" angle irons to act as runners and spacers. The sheet may be any convenient width and length, depending on the nature and size of the work. It is flared out at the upper edge to act as a hopper. The contrivance is placed with the angle irons against

the surface to receive the mortar finish. Mortar is placed on one side and concrete on the other. Then by means of handles the sheet is drawn up and the concrete tamped to unite the mortar and concrete. It should not be drawn quite out of the concrete. There is difficulty in carrying up a long line of this kind of concreting, especially at corners and at the junction of two sheets. It cannot be worked well in a narrow space.

Of the foregoing methods the manipulation with the spade or other similar tool against the forms is probably the most satisfactory for the reasons that it can be done with wet concrete and in a narrow space, and because it results in a more uniform concrete. The mortar of the entire mass is uniform, the only difference at the surface being that the stones are not exposed. Separate mortars do not bond so well. It is especially true of work done in freezing weather that a mortar of a different mixture from that of the concrete is apt to break away from the body.

Concrete of small aggregates, as that used in reinforced concrete, does not need to have the mortar brought to the surface, as a rule. The churning and puddling, which should be done in any case to work out the air bubbles and to work the concrete into corners and around the steel, will serve to give the concrete the smooth surface desired, if the mixture is the proper richness and consistency.

In order to cover up the roughness of the boards and to prevent them from adhering to the concrete, as well as to prevent the knots from discoloring the concrete, a filling coat is sometimes used on the wood. Soft soap may be used for this purpose, applied with a brush. Linseed oil may also be used. Fatty oils should not be used, as they act on fresh concrete disintegrating and discoloring it. Hot paraffine is sometimes used. Crude oil is a very good substance to prevent adhesion of the concrete to the wood. A mixture of crude oil and kerosene also *gives good results.*

If the wood is thoroughly wet with water before the concrete is laid, there is not much danger of concrete adhering. This wetting is to be recommended for the further reason that the wood will not then absorb water from the concrete.

Paper, unless it is oiled, will stick to the concrete and is hard to remove. Burning may have to be resorted to.

One method used successfully to cover up the grain of the wood was to paint the surface with a gloss oil and to blow sand into this with a bellows.

The author does not know of any case where canvass painted with linseed oil has been used as a cover for rough forms, but he believes it would be an admirable material for the purpose. It is waterproof and would therefore not absorb water from the concrete; it would also prevent the leaking of the liquid mortar that occurs at cracks or joints in the wooden forms. It would probably be economical for the reason that it does not require planed boards in the forms and it could no doubt be used repeatedly. It would further help to keep the frost out of the concrete.

With all the means used to cover up the irregularities of the wood and to make the surface smooth there will still be some roughness not commercially avoidable. Some treatment after removal of the forms is generally necessary. Air pockets may occur in places where the holes will be exposed. These should be plastered with a rich mortar. Corners may break off in removing the forms. These should also be plastered. If large chunks of concrete break away in an important part of the structure, the best thing to do may be to remold the piece. Any sign of extended weakness in the concrete may show that a bad batch of concrete was used or that the concrete has been mistreated during setting.

If made right, the concrete surface will have a skin of neat cement. It is generally desirable for appearance sake to remove this, and there are several ways to do it, depending upon the length of time that the concrete has set, before it can be made accessible for treatment. The time

that elapses from the placing of the concrete until the surface can be exposed depends upon the kind of concrete and the nature of the part of the structure in question.

If the surface can be exposed a few hours after the concrete is placed, this cement may be removed with clean water applied by means of a hose. The hose should be used without a nozzle, as the pressure would gouge out stones. Water may also be applied with buckets. Heavy walls in dry concrete could probably have the forms removed in half a day or so after concrete is placed, and the surface could be thus treated. The forms should not be removed so soon on a high section of wall.

If the concrete has set for about 24 hours, clean water and scrubbing brushes will remove the outside skin of cement. If about two days have elapsed, wire brushes may be needed, using water to wash away the loosened cement. If the concrete surface is hard, more vigorous work is required to make it smooth and to take off the skin of cement. Blocks of sandstone, or of concrete of cement and sand, or of carborundum, with water, may be used to scour the surface. These are rubbed with a circular motion. When a sand blast is available, this is an excellent means of accomplishing the desired result.

Some preliminary treatment will usually be found to be necessary, such as chipping off rough projections, as those left by cracks in the mold, filing off the arrises, etc.

These scouring processes have for their object the securing of a smooth surface. Sometimes, after the surface is washed and scoured reasonably smooth, some grout of cement and sand is brushed on, and by the same circular motion with the bricks, this grout is worked into any pores in the surface. The result, after the setting of the grout is a very smooth surface.

A smooth surface is not always desirable. Some roughness is more in keeping with the nature of the concrete and is more pleasing in appearance in many situations. The washing or scrubbing off of the skin of neat cement, *above described, will* expose the surface of the aggregate

and leave the desirable dull or rough surface without any further treatment. The appearance will then depend upon the selection of the aggregate. If a mortar of sand and cement be exposed on the finished surface, the washed surface will resemble sandstone. By using white sand the appearance will be that of nearly pure white sandstone. Other colors can be obtained by using different colored sands.

Torpedo sand, a sand having large grains, used in the surface mortar, will, when the surface is washed clean of the cement skin, give a good appearance. The same is true of small regular sized pebbles or of small sized crushed granite or limestone, screened to ¼ in. or so. Colored granite can be used with good effect to obtain a red, black, or gray surface. Colors obtained in this way are more durable and uniform than those made by use of coloring pigments.

The surface treatment by use of regular sized particles in the aggregate and subsequent washing off of the skin of cement may be carried to any size of stones, even to that of cobble stones. This style of finish in pebble size is especially appropriate in park pavilions, concrete fences, etc. If a dense impervious concrete is not essential, a dry mixture can be used and the washing away of the mortar skin dispensed with. This sort of finish on reinforced concrete arches can be employed by using the dry concrete with coarse sand, or small pebbles, or ¼ in. broken stone for a depth of an inch or so against the forms and a wet impervious concrete surrounding the steel.

Where the concrete is molded and not plastered, there is an advantage in using a stiff mixture in preference to a wet mixture, as the forms can be removed in a shorter time, and the washing off of the cement is easier and cheaper of accomplishment than when it has a harder set. The lack of density and impermeability due to the dry mixture would make this method less applicable to buildings whose character demands that the walls be not absorbent of water.

Concrete of materials not affected by acids, such as sand, gravel, granite, and trap can be treated to a surface scrubbing of a 20 per cent, solution of hydrochloric acid. This will remove the cement skin, even after a hard set. The acid must be immediately neutralized with alkali to prevent penetration into the concrete, and all must be washed off with clean water. The disadvantages connected with this method are the high cost and the difficulty of handling the acid to apply it and the fact that the acid that wastes and is not neutralized may penetrate into the base of the structure and destroy the concrete.

This acid wash may also be used to remove efflorescence from concrete surfaces.

Coloring of the surface of concrete may be effected, as stated, by using naturally colored aggregates and washing the surface. Pigments are also used. These are not apt to be very permanent. If used in large quantities they will weaken the concrete. If used in the entire body there is a waste of material in that which is not exposed, and it is difficult to get uniformity by any other means, unless it is a case where plastering is permissible. If pigments are used, it is best to mix them thoroughly with the dry cement. A little lamp black can be used to advantage in ordinary concrete to relieve the dirty color of the concrete. This could be placed in the mixer with the cement, a given quantity for each bag of cement.

Concrete surfaces will not hold oil paint very well; the washing off of the skin of neat cement will make them more retentive. In any event oil paint is not appropriate to the nature of the concrete except for interior walls and ceilings. A wash of neat cement can be applied to a wall, or the cement may be mixed with plaster of Paris or better with marble dust. Either of the latter will give the appearance of marble. In applying these the mixture should not be too thin or it will crock. If it is too thick it cannot be worked with the brush. An ordinary whitewash brush is used in its application. It is necessary that the surface be thoroughly drenched with water just before

he application, otherwise the wall will absorb the water n the cement and prevent setting. The surface must be :ept wet for a day or so.

Some of the things that contribute to unsightly appear- nce in concrete work are the following. Their remedy s evident. (1) Irregularity in the nature of the ingre- lients. This applies to the stone, sand, and cement. It is vident that unless the first two run uniform, there will not e a uniform surface on the concrete. It is also true that lifferent brands of cement may give different colors. (2) ack of uniformity in the amounts of ingredients in each atch. (3) Insufficient mixing in any or all batches of oncrete. (4) Dirt on the forms. (5) Want of care in lacing, tamping, spading, etc.

Of surfaces that face upward, vertical surfaces, and hose that face downward the latter are the most difficult of treatment. This is of course because of the fact that generally from two to four weeks must elapse after plac- ng the concrete before these can be safely exposed, because of the necessity of supporting the setting concrete. The lifficulty of reaching such surfaces makes their treatment loubly expensive. A sandblast or scouring with bricks for nterior exposed beams and ceilings would be best. For- unately in arches and exterior girder work the under side does not show up to any extent and can often be left rough.

Pavements and floors are in a class by themselves and offer the most satisfactory conditions for surface finish. Ordinary sidewalks as commercially made are often put down in the following way. After the sub-grade is tamped and the layer of cinders or broken stone for drainage is laid down and tamped (generally a thickness of 4 to 6 inches) a layer of stiff concrete is placed and rammed. This is about 3 or 4 inches thick and is usually a gravel concrete of about 1:3:6 mixture. This concrete base is blocked off and sand joints used to separate the blocks. Then this concrete is left until the next day, when a top coat of cement *mortar is laid,* usually a 1:2 or 1:3 mixture

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ing is that the skin of cement thus formed shrinks in setting and causes hair cracks or mapping of the surface. Sometimes large shrinkage cracks are formed, which continue to increase in size. Lack of proper provision for expansion and contraction at the regular joints intended to take up the motion may be the cause of some of the large cracks observed. It is not enough to make a groove at the surface of the pavement to relieve the tension. There should be a complete separation of the blocks.

The splitting away of the mortar coat is the natural result of insufficient bonding between the concrete base and the mortar finish.

In contrast with the method of laying pavements above referred to the author recently observed the laying of concrete in the floor of one of the largest reinforced concrete buildings in the world. While the body of the concrete was fresh, the surface being quite rough, no attempt whatever having been made to make it even approximately horizontal, the mortar for the top finish was poured. This was of the consistency of soup. Men waded in concrete 6 or 8 inches deep to spread the mortar about with shovels. Then the top surface was leveled off by working a straight edge back and forth. In this work the concrete of the body of the floor slabs while it was soft enough to allow the men's feet to sink into it, did not show any water on the surface for the reason that it was not tamped.

If there is an excess of water in the concrete base in pavement work the bond may be impaired between this and the finishing mortar. Concrete in the base almost wet enough to flow, spread out to a uniform depth with rakes rather than tamped to a smooth flat surface covered with laitance, would present a much better opportunity for the bonding of the finishing mortar.

In steps and other parts where vertical surfaces must be finished off it is necessary in the ordinary methods of making sidewalks, to use *stiff* concrete and to plaster on the *mortar, also a stiff mixture*, as soon as the molds can be *removed. The necessity* for using stiff concrete is to allow

of early removal of the molds, so that plastering can be done on the surface before the concrete has a hard set. Generally steps are troweled to a smooth finish, though the treads would be better to have a rough surface to prevent slipping.

The rough surface on pavements, so desirable on steep grades and in fact desirable most anywhere to overcome slipperiness in cold or muddy weather, is obtained in several ways. A steel trowel must not be used in the finishing process, if any kind of rough surface is to be made. In fact a steel trowel should be used sparingly in pavements. If a smooth glossy surface is wanted, it should be made by using a rich mixture and fine sand in the finishing mortar rather than by rubbing with a steel trowel.

In the process of leveling off the pavement when the mortar coat is laid a wooden straight edge is used. This is worked back and forth on the side boards forming the mold for the pavement until all of the high places are brought down and all of the hollow places filled in. If the mortar is uniform in consistency and well mixed, the work could stop here, so far as the greater part of the surface is concerned, and the rounding of corners and marking and dividing into blocks would complete the pavement. A strip about 1½ in. wide rubbed smooth with a tool for the purpose is usually made around each block. The surface can be given a kind of regularity, if in manipulating the straight edge it be moved back and forth two or three inches as it is pushed forward making sinuous lines along the pavement. A wooden float may be used, after the surface is made horizontal by the straight edge or by a trowel, where a straight edge cannot be employed. The desired roughness can be made in this way. What is probably the most pleasing surface appearance for pavements is obtained by use of a wooden plate. The surface is stippled with this, and by suction against it a most workmanlike and artistic finish results. Another way to produce the rough surface is to throw the last half inch or so of mortar with the trowel, a little at a time.

It is a mistake to sprinkle dry cement on the surface of a pavement.

To overcome the tension due to troweling, where the pavement is troweled, and to roughen the surface it may be broomed or brushed with a corn brush just after troweling.

Where a smooth troweled surface is desired the final rubbing should be done a half an hour or so after the mortar is laid. Less cement would then be drawn to the surface, and the surface tension will be less. For smooth troweling the mixture should not be very wet.

It is best to lay pavements in alternate blocks against bulkheads or vertical boards placed temporarily where joints are to be. Where this is not done, the concrete base should be marked off in blocks and separated, while the concrete is green, by making joints ¼ inch wide or so, filled with sand or tar paper or other filler. The location of these must be marked before the mortar finish is laid, and when the latter is finished off it should be cut with a thin trowel making a complete separation and not merely a groove. Very little if any space is needed between blocks to take up expansion and contraction due to change in temperature, but the contraction due to shrinkage in setting may be enough to split the block at some other section than in the groove, unless a complete separation is made.

Pavements that are not on a grade are generally given a slope of ¼ in. to the foot for drainage.

There is a difference of opinion as to the proper mixture for a mortar finish on pavements. Some engineers would not use a mortar richer than 1:2 or 1:3 because of the fact that with troweling a rich mortar will check. Others specify mortar coats as rich as 1:1. If the mortar is of the same richness as that used in the concrete (e. g. a 1:2 mortar for a 1:2:4 concrete) there ought to be equilibrium in the mass. Mixtures of 1:1½ to 1:2½ with granite screenings *that will* pass through a sieve having *¼ in. meshes give satisfactory* results. The former is *much used in high* class pavement work. Particularly

[illegible]y s[illegible]s are obtained by richer mixtures, using as [illegible] as [illegible] Such pavements, while they may be good in the interio[illegible]f buildings, are too slippery for out-of-doors.

For har[illegible]nooth surfaces fine sand may be used. Fine sand shoul[illegible]nly be used in a rich mixture. For a rough surface or [illegible] leaner mixtures use coarse sand or screenings.

Plaste[illegible] on vertical surfaces of concrete should be avoided [illegible] practicable. It is often necessary, however, and whe[illegible] is done, special precautions should be taken to make [illegible] is not important that the body of [illegible] impermeable, plastering will adh[illegible] be made dry, on account of the rou[illegible] a further advantage in using dry concre[illegible]lerations do not demand wet concrete, namely; t[illegible] can be removed soon after the concrete is placed an[illegible]before the cement has a hard set. It would be well in s[illegible]e cases to wash or scrub the surface or to pick it before applying the plaster. Lime paste mixed with the cement and sand for plastering increases the adhesion and lessens the tendency for hair cracks to form on the surface. Lime delays the setting of the cement and makes it easier to work. Lime paste can be used in quantities up to an equal amount with the cement. If a hard surface is desired, less lime would be used. In this plastering, as in pavements, it is well to go over the surface with a brush or broom after troweling to relieve the tension of the cement. In plastering piers and abutments the trowel marks can be removed by brushing horizontally with a whisk broom. The surface appearance is greatly improved by this means.

A simple and inexpensive mode of plastering is what is called a splatterdash coat. The mortar is thrown or splashed against the surface with a paddle. An effective rough surface can be made in this way. The methods used on pavements to produce a rough surface, by means of wooden floats and stippling, can be employed to good advantage on walls. A rough surface is generally bette

appearance and less liable to crack than a smooth surface.

A method recommended to secure a good bond is as follows. First the surface is washed, then after thorough drenching with water a coat of neat cement grout is brushed on. While this is still wet a coat of plaster about ¼ in. thick is put on. Succeeding coats of the same thickness are applied about an hour apart until the desired thickness is attained. As a finish the last mortar may be thrown on, to give a rough surface.

Plastering should, in general, be resorted to only to fill in holes and to smooth over rough places. A plaster coat should either be very thin, that is, just enough to fill irregularities, or it should be one to three inches thick, so that it will have some strength in itself.

In making steps the forms should be of planed boards. The mortar of the concrete should be worked up against the forms in a manner heretofore described. Upon removal of the forms the surface can be made smooth by plastering up any holes with a 1:1 or 1:2 mortar and by using the same in a wash over the surface, rubbing it smooth with a wooden float or with a scouring brick.

In sea walls it is well to have as smooth and dense a surface as possible. This is to prevent penetration of the salts.

It is said that mortar that has set up for 3 or 4 hours will adhere better as a plaster and shrink less than freshly mixed mortar.

On a dry, hard surface a plaster coat or floor finish should be 2 or 3 inches thick.

On a brick wall or on cinder concrete nails can be driven in to anchor the plaster, or expanded metal or other metal lath can be nailed on and plastering done on this base. Old brick buildings can by this means be given a cement finish, which, when properly done, enhances their appearance very greatly.

Old wooden floors can be covered with a finish of ½ in. *or more of a* concrete made with saw dust as one of *the aggregates. This, too,* is anchored to the floor by

means of nails driven into the wood. The use of this kind of floor finish on new concrete floors has not received the attention and trial it deserves, possibly on account of the lack of workmen skilled in its manipulation.

Common sense and artistic judgment are probably violated more in the matter of the finish for cast blocks than in any other line. There are probably more "rock face" blocks made than any other kind, and there is nothing more absurd than an imitation of rock faced stone blocks in cast stone work. Cast stone or concrete blocks can never have any standing so long as their very appearance holds them out to be what they are not. It is legitimate to tool dress or finish concrete blocks or artificial stone, but it is not legitimate to cast in them an imitation of a dressing which they do not receive. Rock faced is the name given to the finish of stone that has been rough dressed. The surface is the natural fracture of the stone. No mason would attempt to produce this appearance by carving the entire surface. It would not be economical in concrete blocks to break off enough of the surface to produce this effect, even if the concrete should possess the property of fracturing in this way. Its sole beauty lies in its irregularity and in the fact that it is a natural and not an artificial result. In concrete blocks regularity is necessary and painfully manifest. It would be as rational to carve a human face in the rugged side of a mountain of rock to enhance its beauty as it is to cast a rugged face in a molded block.

A very satisfactory surface is obtained in concrete by tooling. For rough work a pick may be used to break off the outer skin. Light picks may be employed on the surface while the concrete is rather green. A cheap and effective finish can be obtained in this way. This is especially suitable for factory buildings. The picking is done, lightly, and it is said that a workman can finish 1,000 sq. ft. a day. A pointed chisel may be used for the surface of heavy walls. Smaller work may be crandalled. Bush hammering is another method employed; this may be done by unskilled workman. To avoid a broad expanse of flat

surface the wall should be laid off in panels. A good way is to divide the surface into blocks by nailing triangular strips on the forms. The V shaped grooves would not be dressed, but only the flat part.

For chisel dressing allow ¼ to ⅛ in. to be removed by the tool. Bush hammering on a good flat surface need not cut in any measurable depth. All arrises should be rounded to about ⅜ in. radius. A file can be used for this purpose. Dressing with hammers should not be done before the concrete is a month or two old. Small stones are apt to be dislodged by the process, unless they are firmly cemented in.

On some kinds of artificial building stones a tooling machine of carborundum wheels is used. This can be used on very green concrete, as it grinds soft and hard spots alike and does not jar the stone. The surface finish is very good.

Where compressed air is available, pneumatic tools for crandalling or otherwise dressing the concrete surface would be economical and satisfactory.

It may be found difficult to dress the surface of a concrete made of a hard gravel. The stones may be gouged out of the cement before they will break.

Forms for Concrete Work.

Forms should be designed so that they will retain their shape, until the concrete has hardened sufficiently to be self supporting. Hemlock should not be used, except in the roughest kind of work. Spruce and pine are preferable. Kiln dried lumber or thoroughly seasoned lumber is not desirable, because the dry wood will swell. Green wood is also undesirable on account of liability to check and warp. Partly seasoned lumber is best.

Whether or not lumber should be planed will be determined by the kind of surface desired. Dry concrete does not require tight forms, since there will be no liquid mor-

tar to run out of the cracks. For wet concrete the forms should be as tight as possible. Cracks may be filled, where necessary, with putty or clay or plaster of Paris. In order to prevent excessive pressure and heaving of forms due to swelling the boards can be planed beveled on one edge, the full width being against the concrete. The narrow edge of a board will then be brought in contact with the wide edge of the next. Closer fit can also be secured.

For fine work it would be well to shellac the wood to prevent penetration of water and consequent swelling. If end grain is in contact with concrete, it should be covered with shellac to prevent absorption of water.

Tongued and grooved boards are best for a smooth surface, though they are more expensive. Boards surfaced on one side and on the edges are good enough for most work.

The thickness of boards used for sheathing or lagging should be such as to keep deflection within limits. The thickness will be determined more by the amount of deflection than by the extreme fibre stress produced by the load. In general, to avoid springing, 1 in. boards should not be used on wider spacing than 2 ft. between studding. The span for 1½ in. plank should be not more than 3 ft. and that for 2 in. plank not more than 4 ft. for vertical loading. Wider spacing than this can be used, where the pressure is lateral, as in walls up to about 4 ft. for 1½ in. plank and 6 ft. for 2 in. plank.

For the under side of floor slabs generally one inch boards are used, with studding 16 to 24 in. apart. For columns and the sides of beams one inch boards may also be used running lengthwise and held in place by outside forms made of 2 in. x 4 in. or larger sized pieces. These may be spaced at intervals of about two ft. and held in place by means of bolts in the case of columns or be wedged against those of the next beam in the case of beams. A heavy plank along the bottom of a beam would serve to support the weight on the shores and allow the removal of the side sheathing in advance of the bottom supports.

Nailing should be reduced to a minimum
ocking off the forms
ete, also the wood will be
e by repeated nailing
nere possible to hold the
Thick plank will be
ards, if used many times
tter and last longer
ick lagging, say 2 in.
Cutting of the wood
ble.

Where there is much
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large. Designing the floor so that the panels are uniform in size will greatly facilitate the construction.

Beams and girders should be given a small camber, say an inch for every 20 ft. of span.

Wires are very often used to hold forms against spreading. These can be cut off with cutting pliers at the surface of the concrete, or, to prevent rust stains, they may be cut with a chisel below the surface and the hole plastered up. Bolts are sometimes used for the same purpose and removed with the forms. To prevent adhesion to the concrete these may be run through gas pipe or tin tubes, which remain in the concrete. If the gas pipe or tin tube is brought out nearly to the surface and the end stuffed with cloth or waste to prevent admission of concrete, the bolt may be drawn out. The waste packing can be removed and the hole plastered up.

Wooden blocks used as spacers must be removed as concrete is placed. No wood should be left in concrete, either by accident or for any structural reason. Wood buried in concrete will suffer dry rot, as it cannot season.

Tie rods or bolts should not be close to the edge of the concrete, as they are liable to break out pieces of concrete in removing.

In sewer work for sewers of sufficient diameter for men to work in, it is a good plan, as stated elsewhere, to omit lagging on the invert up to a point where it is needed to prevent flowing of concrete. This affords better opportunity to place and compact the concrete as well as to trowel the surface to a smooth finish. The ribs would be in place just the same to guide in forming the sewer. Loose lagging can be used on the upper part of the sewer. This allows easy removal as the work advances. Loose lagging on culverts and arches is often desirable.

Vertical props should, in general, be braced together. This prevents accidental dislodgement and prevents swaying of the structure. The props or shores should rest on *solid footings* and should be so designed that they can be *the last pieces* removed and that they can be left in place

ter the concrete is self supporting. Such a condi- ›ften desirable in a building where a floor is support- rops to a floor below, the latter being just capable of ›port. It may be necessary to allow the shores to in the successive stories down to the foundation, r not to strain these floors with load they are not d to carry, until the top floor is self supporting. should be props under the girders close to columns o relieve the green concrete of the columns of the of the floors.

s should have double wedges of hard wood under so that they can be removed without jarring the :e and so that they can be eased off without removal, it is not positively known that the concrete is thor- hardened. Very heavy posts as those supporting pan arches could rest in sand boxes arranged with o let out the sand and lower the posts by this means. a unit compression on posts 1,000 lbs. per sq. in. ›mmended for pieces whose unsupported length is nes the least width or less, and 400 lbs. per sq. in. eces whose unsupported length is forty times the width. For other ratios proportionate values are to ›d. Lengths more than 40 times the least width l be avoided.

beams a unit of 800 lbs. per sq. in. is recommended. span in feet be divided into the coefficient C, in the ·ing list of common sizes, the result is the total weight ınds that the beam will carry as a uniformly distribut- .d.

2 x 4,	C= 2840
2 x 5,	C= 4440
2 x 6,	C= 6400
2 x 8,	C=11400
2 x 9,	C=14400
2 x 10,	C=17800
2 x 12,	C=25600
3 x 14,	C=52270

3 x 15, ...	C=60000
4 x 16, ...	C=91020

As an example, suppose it is desired to know the s of joists required to carry a 6 in. slab of concrete on joi spaced 18 in. apart, the span being 10 ft. and an acciden load of 75 lbs. per sq. ft. being provided for. The to load carried by a joist is (75 + 75 or 150) x 10 x 1.5 2250 lbs. Reversing the rule above given and multiplyi this by the span length we have C = 22500. Hence 2 x joists would be needed.

The depth of beams should generally be between o tenth and one-twentieth of the span. Beams deeper th one-tenth will be overstressed in shear, when strained their capacity in bending; beams shallower than ab one-twentieth will deflect too much under load.

In proportioning the forms allowance should be ma for accidental load that may be placed upon the floors the processes of construction. A uniform load of 75 l per sq. ft. for slabs and of 50 lbs. per sq. ft. for beams a girders ought to cover this contingency, as well as t weight of forms, except where the forms are unusua heavy.

The lateral pressure of the green concrete may be tak as equal to that of water, keeping in view the amount concrete that will be placed at one time.

There should be a door at the bottom of column for to allow cleaning out before concrete is placed. Also t bottom board in wall forms and girder forms should placed last where practicable.

Collapsible forms for tunnels, culverts, and sewers often made use of. Sheet steel is sometimes used in pl of lagging of wood. Forms that are not collapsible m often be removed by revolving about the vertical ax then if the lagging is loose and not nailed, the form readily be removed and placed forward.

A good and economical form for circular columns be made of sheet steel. It may be made of two semicir lar sections, flanged and bolted together.

In bracing forms wooden pieces should be used, generally 2 x 4 scantling. Wire ropes should not be employed for this purpose in any case, as the stretch in these may be disastrous to the structure.

In heavily battered walls, as the wing walls for an abutment, a cover on the sloping part of the wall is generally necessary. This should be removed as soon as possible so that the top surface can be plastered and made impervious. If much wet concrete is poured at a time in such wall, it may be necessary to tie down the cover of the sloping portion to prevent its being lifted by the pressure of the concrete. It could be anchored by wires into the body of the wall. If secured only to the side forms, the pressure might lift these.

In footings and retaining walls, instead of having a heavy batter, steps are usually preferable, with the tread sloped so as to shed water. This is especially true when dry concrete is used, as it allows access to the concrete for tamping.

Lateral pressure of wet or of tamped concrete must not be overlooked in the building and bracing of forms. The sides of beam and wall forms should be held against springing by bolts and wedges, where practicable, or by stiff braces.

A suggested scheme for slab and beam forms is as follows. Use a heavy plank under the girder 4 in. wider than the girder. On top of this nail a one inch planed board, the width of the girder, along the middle of the plank. There will be a 2 in. margin on each side. One inch of this will be taken up with the side boards for the girder. The other inch will be used as a ledge to support the joists for the slab which can be blocked up by nailing them to the vertical studding for the sides of the girder. If strips be nailed lightly to these joists and studding, and if the sheathing for slabs and girders be nailed together with cleats to form units, there will need to be but little nailing into the wood in contact with the concrete. The heavy plank under the girder should be supported by props 3

to 6 ft. apart. For heavy girders it may be stiffened b knee braces to the props or by a pair of joists nailed to th side of the props. By the above described arrangemen the forms under the slabs and on the sides of girders ma be removed and used elsewhere while the weight of th girders is still on the shores.

It is important that columns be plumb and that beam be straight and true to line. Before beginning to place th concrete all forms should be brought to correct positio and true to line and securely braced. Warped and incline surfaces of walls, intended to be plane and vertical, an wavy copings are very unsightly.

Sharp corners in the concrete should be avoided. Th arrises should generally be either chamfered off in th mold or trimmed rounding when the forms are removed.

In dividing a wall into blocks the joints should be ' shaped, either sharp or truncated. The recess should nc have parallel sides, as the withdrawing of the moldin pieces would break off spalls of concrete; also the con crete will not as readily fill in around the squared reces The same is true of moldings. These should not have re cesses with parallel sides. Surfaces that face upwar should in general, have a slope outward so as not to bin on the forms and so as to allow concrete to be more easil worked under the form. Further, such surfaces will tur the water and absorb less.

The design of the work should be made with a view c reducing to a minimum the difficulties inherent in the hand ling and placing of concrete and with an appreciation c the skill of the workmen executing the plans. Molding should be as simple as possible and in conformity with th construction. Small arches and buildings should hav light moldings and ballustrades, where any are used. Mas sive structures should have bold and massive ornamen tation. Fine details should not be attempted on out-doo work. These can be executed by use of lime or plaster c Paris in conjunction with cement for indoor ornament n. Imitations should be avoided. Broad plain surfa

should be broken up by paneling where practicable; the heavier the work the more massive and bold the paneling should be.

The Properties of Concrete

Concrete partakes of the properties of the stone or other aggregate from which it is made, though these properties are affected by the mortar with which the aggregate is cemented together. The property of greatest importance is the compressive strength. The compressive strength of the concrete is in general less than that of the stone of the aggregate. The unit compressive strength tested in cubes is generally less for small cubes than for large ones. It is greater for flat discs than for cubes, the flatter the disc the higher the unit.

On account of the many factors that enter in the manufacture of concrete affecting its uniformity, and on account of the very nature of concrete, being a non-homogeneous substance, laws that are even approximately exact cannot be written that will tell the compressive strength without allowance for a liberal variation. If the dimensions of the specimen were greatly in excess of the largest piece of the aggregate, unit results would no doubt be more uniform. Sand and cement in ordinary sized specimens give more uniform units than concrete, because the grain of sand bears so small a ratio to the size of the specimen that it is possible to test.

The best that can be done in the matter of establishing a unit value for concrete in compression is to take average values of a large number of tests made under different conditions, and in design to use a liberal factor of safety to cover irregularities and imperfect work. A standard concrete should be used in reinforced concrete, and uniform conditions should be striven for. The cube is probably the best standard for compressive strength, and its *size should be from 8 to 12 in. so as to minimize the effect of non-uniformity.*

S ON CONCRETE CUBES

Aggregate as stated in 1st. Column.
Compressive Strength in Lbs. per Sq. Inc

I. MORTAR - 1 PORTLAND CEMENT : 1 SA

Aggregate	Kind of Stone	Age of C	No. of samples	Average Compr. Strength	Authorit
1½	Trap	11 to	4	3944	U. S. 1898
3	"	7 "	9	2442	" "
3	"	19	9	3346	" "
3	"	31	10	3676	" "
3	"	61 "	7	4591	" "
3	"	4 mo.	6	3079	" "
3	Pebbles	7 to 11 da.	4	2092	" "
3	"	21 " 26 "	4	3052	" "
3	"	29 " 46 "	5	3642	" "
3	"	61 " 70 "	3	3793	" "
3	Cinder	29 " 39 "	18	1544	" "
3	"	90 " 102 "	15	2186	" "
3	2 Trap, 1 Gravel	7 da	1	1800	" "
3	" "	23 "	1	3806	" "
3	" "	32 "	1	5024	" "
3	" "	67 "	1	4700	" "
3	2 Pebbles, 1 Gra	7 "	1	1486	" "
3	" "	22 "	1	2676	" "
3	" "	32 "	1	3000	" "
3	" "	65 "	1	3800	" "
3	1 Trap, 1 Pebbles 1 Gravel	7 "	1	1520	" "
3	1 Trap, 1 Pebbles 1 Gravel	22 "	1	2800	" "
3	1 Trap, 1 Pebbles 1 Gravel	32 "	1	2700	" "
3.9 to 6.5	Broken Stone	19 to 22 mo	64	3334	Rafter[3]

TESTS ON CONCRETE CUBES

Continued

MORTAR - 1 PORTLAND CEMENT : 2 SAND

Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
Trap	3 mo.	8	1984	U.S. 1899
"	4 "	4	2834	" "
"	3 "	10	2030	" "
"	4 "	5	2809	" "
Gravel	28 da.		2125	Dyckerhoff
Cinder	39 "	3	1098	U.S. 1898
"	102 "	3	1634	" "
Trap	3 mo.	10	1957	" "
"	4 "	16	2244	" "
Granite	28 da.	2	1607	" 1899
Pebbles	3 mo.	5	3206	" 1898
Conglomerate	7 to 10 da.	25	1565	" 1899
"	1 mo.	30	2399	" "
"	3 "	30	2896	" "
"	6 "	26	3796	" "
Slag	52 da.	8	3392	" 1900
"	85 "	8	4135	" "
Cinder	38 "	3	904	" 1898
"	98 "	3	1325	" "
Brick	28 "	6	720	" 1899
"	3 mo.	5	2757	" "
Limestone	6 da.	4	1084	" "
"	11 to 13 da.	8	1676	" "
"	19 " 20 "	6	2040	" "
"	52 da.	5	3604	" 1900
"	85 "	5	4256	" "
Trap	3 mo.	12	2270	" 1899
"	4 "	6	2321	" "
Gravel	28 da.		2387	Dyckerhoff
Cinder	29 to 38 da.	15	724	U.S. 1898

TESTS ON CONCRETE CUBES
Continued

II. MORTAR - 1 PORTLAND CEMENT : 2 SAND

Aggregate	Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
5	Cinder	90 to 101 da.	15	1081	U. S. 1898
Voids filled	Gravel	1 to 2½ mo.	7	1477	Baker
	"	5 " 8 "	2	1742	"
	"	12 " 15 "	2	4583	"
	"	20 " 30 "	6	2278	"
	"	32 " 36 "	3	2365	"
	"	35 " 36 "	2	3497	"
6	Trap	3 mo.	13	1869	U. S. 1899
6	"	4 "	7	2411	" "
6	Gravel	10 da.		694	A. W. Dow
7	Trap	3 mo.	15	1466	U. S. 1899
7	3 Gravel, 4 Stone	6 "		2000	Baker
7	" "	32 to 37 mo.	6	4428	"
8	Trap	3 mo.	18	1163	U. S. 1899
9	"	3 "	10	816	" "
5.18 to 8.93	Limestone	19 to 22 mo.	128	2678	Rafter

III. MORTAR - 1 PORTLAND CEMENT : 3 SAND

Aggregate	Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
5	Gravel	28 da.		1682	Dyckerhoff machine mixed
5	Limestone	3 mo. 2 da.	8	4043	U. S. 1900 hand mixed
5	"	" "	4	3187	U. S. 1900
6	Trap	8 da.	2	858	" 1898
6	"	17 to 20 da.	3	1507	" "
6	"	42 da.	3	2192	" "
6	"	4 mo.	15	1648	" "
6	Gravel	45 da.		1628	A. W. Dow
6	"	3 mo.		2671	"

TESTS ON CONCRETE CUBES

Continued

1 MORTAR. 1 PORTLAND CEMENT : 3 SAND

Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
Gravel	6 mo.		1844	A. W. Dow.
"	1 year		2824	"
"	28 da.		1515	Dyckerhoff
Conglomerate	7 to 10 da.	23	1420	U. S. 1899
"	1 mo.	28	2174	" "
"	3 "	30	2522	" "
"	6 "	23	3110	" "
Cinder	29 da.	3	529	" 1898
"	91 "	3	788	" "
Brick	3 mo.	7	2207	" 1899
2 Grav. 4 Brok. sto.	1 year		2835	A. W. Dow.
3 " 3 " "	10 da.		949	"
3 " 3 " "	45 "		1854	"
3 " 3 " "	6 mo.		2070	"
3 " 3 " "	1 year		2751	"
Limestone		10	3067	Baker
"	7 da.		1584	125% voids filled
"	10 "	1	908	A. W. Dow.
"	30 "		1795	125% voids filled
"	45 "	1	1791	A. W. Dow.
"	90 "		2579	125% voids filled
"	3 mo.	1	2256	A. W. Dow.
"	6 "	1	2511	"
"	1 year	3	2442	"
"	7 da.	1	1282	Voids filled
"	30 "	1	1672	" "
"	90 "	1	1128	" "
"	19 to 22 mo.	128	1872	G. W. Rafter
"	7 da.	1	892	75% voids filled

TESTS ON CONCRETE CUBES
Continued

III. MORTAR - 1 PORTLAND CEMENT : 3 SAND

Aggregate	Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
10	Limestone	30 da.	1	472	75% Voids filled
10	"	90 "	1	919	" " "

IV MORTAR - 1 PORTLAND CEMENT : 4 SAND

Aggregate	Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
8	Trap	4 mo.	10	1080	U. S. 1898
5	Gravel	28 da.		1273	Dyckerhoff
8½	"	28 "		1204	"
10 to 13	Limestone	19 to 22 mo.	10	1735	G. W. Rafter

V. MORTAR - 1 PORTLAND CEMENT : 5 SAND

Aggregate	Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
10	Trap	4 mo.	12	708	U. S. 1898
11.25 to 15.41	Limestone	19 to 22 mo.	32	1554	G. W. Rafter

VI. MORTAR - 1 PORTLAND CEMENT : 6 SAND

Aggregate	Kind of Stone	Age of Cube	No. of samples	Average Compr. Strength	Authority
12	Trap	4 mo.	14	625	U. S. 1898
12	Conglomerate	7 to 10 da.	25	602	" 1899
12	"	1 mo.	30	993	" "
12	"	3 "	30	1078	" "
12	"	6 "	21	1408	" "
16.66	Limestone	19 to 22 mo.	4	1427	G. W. Rafter

When exposed to ordinary conditions of weather, 1:2:4 concrete will attain in two or three months a unit strength of 2,000 to 2,500 lbs. per sq. in. Carefully treated specimens placed in water, or kept under cover and damp, attain strength of 2,500 to 3,500 lbs. per sq. in. or even 4,000 lbs. It is not possible to realize this latter in ordinary work. The former represents more nearly average conditions in a structure.

The table of tests on concrete cubes in the five pages given herewith are taken from Municipal Engineering, January, 1903. The specimens were nearly all 12 in. cubes.

The strength of concrete in cubes has only limited application in the proportioning of the parts of a structure. In plain concrete columns of even a few diameters in length, and in columns having small longitudinal steel rods, a safe unit, if based on the strength of a cube in compression, should have a large factor of safety to cover the uncertainties. A factor of safety of eight or ten for concrete in such condition would be about equivalent to a factor of safety of four for concrete confined as in a reinforced concrete beam or slab, using in each case the unit determined by the cube as a basis. Carefully prepared specimens loaded exactly centrally, as near as it is possible to effect this condition, are not safe guides to the design of members made by unskilled labor, whether or not that labor has competent supervision, when the facts are such that a little variation from perfect conditions makes a great variation in the unit stress. In beams and slabs the concrete under stress is confined and braced in such a way that a bad batch of concrete or poor work has not a very detrimental effect on the compressive strength. Many designers fail utterly to appreciate the fact that in columns of plain concrete, or concrete and longitudinal rods, the concrete is under entirely different conditions than in a beam or slab, and that these unfavorable conditions in the concrete column are in parts of the structure whose failure *means a menace to the* entire structure.

The tensile strength of concrete is from one-tenth to

one-fifth as much as the compressive strength. It is a more uncertain property than the compressive strength. In general the tensile strength of concrete does not enter in the computation of the structural strength of reinforced concrete. It is not thereby eliminated from the problem. It has an effect on the location of the neutral axis, a fact very generally ignored by manufacturers of fine formulas. It has further an important bearing on the calculated deflection of a beam. The tensile strength of good concrete suitable for reinforced concrete, well aged, is from 200 to 500 lbs. per sq. in. If a reinforced concrete beam be designed for 500 lbs. per sq. in. extreme fibre stress in compression, and the concrete would stand 500 lbs. per sq. in. in tension, the beam could be conceived to act for safe loads entirely as a concrete beam. Assuming a modulus of elasticity of an average value, equal for both tension and compression, the deflection could be computed as for a timber beam. For example if the modulus of elasticity is 3,000,000, the deflection at 500 lbs. per sq. in. would be (by the ordinary deflection formula), for uniform load, the square of the span in inches divided by 28,800 times the depth of slab or beam out to out, in inches. If then the depth of slab or beam, out to out, is one-twentieth of the span, the maximum deflection would be the length of the span divided by 1440. If the depth is one-tenth of the span, the maximum deflection would be the length of span divided by 2880.

The foregoing rule will give the deflections that may be looked for in reinforced concrete floors that are carrying their loads safely without cracks on the tension side of beam or slab. This is seen to be small. It will be inversely as the modulus of elasticity. Thus for a modulus of 6,000,000, it would be half as much as for 3,000,000, and for 1,500,000, twice as much. If cracks occur in the concrete, the deflection will be greater. Also if cracks occur, the steel will, at the crack, carry all of the tensile stress. *This is doubtless* what takes place in many beams, namely, *the concrete carries* all or nearly all of the tensile stre

up to the point of its ultimate strength; but there is the possibility, always present, of the concrete cracking and throwing the entire tension on the steel. This assumption of the action of the stresses in a reinforced concrete beam does not carry with it warrant for using any less steel than would be used if the concrete were totally without tensile strength, neither from the standpoint of the unit stress that the steel may take nor from that of the allowed stretch. One crack in a beam will throw all of the stress on the steel, and, if the elongation of the steel is excessive, other cracks would be a natural consequence.

In the matter of the shearing strength of concrete experimenters have reported widely different results, varying all the way from equality with the tensile strength to equality with the compressive strength. This wide variation in the unit shear found by different observers is just what might be expected in a material such as concrete, but the failure to grasp its meaning on the part of engineers generally is not so easily understood. Elsewhere in this book (Shear on Concrete and Its Bearing on the Design of Beams) the author has pointed out that, according to Merriman's well-known formula for combined shear and tension, if the tension on a section in shear be zero, *the tensile unit stress due to shear alone is equal to the shearing unit stress.* It is the simplest kind of reasoning to deduce the proposition that, unless a section in shear is in compression at the same time to overcome the tension resulting from the shear, the shearing strength of the section cannot exceed the tensile strength of the concrete.

The modulus of transverse strength of concrete, or the calculated extreme fibre stress of a plain concrete beam in bending at rupture, cannot be much more than the tensile strength per sq. in. Experimenters have found it to be between one and one and one-half times the unite tensile strength. A safe value, where stone concrete is to be in bending is about 50 lbs. per sq. in.

The modulus of elasticity of concrete is the subject of *this paragraph.* *Modulus* of elasticity is sometimes defined

as the ratio of stress to strain, with stress further defined as an axial load and strain the deformation (stretch or shortening.) The stress would be in pounds per sq. in. and the "strain" in linear dimensions. It is elementary arithmetic that there can exist no such thing as a ratio between pounds and inches, or between any measure of weight and another of length. Even if a liberal meaning is applied to the ratio, it necessitates an explanation as to the terms in which the "strain" must be given. This forced meaning of the word strain is totally unnecessary and does not possess the redeeming feature of being convenient. The language is rich enough in terms to express the particular so-called strain referred to to dispense with a blanket term that must be further defined to make its meaning clear in any particular case. Deflection, for beams, stretch or elongation, for tension members, and shortening, for compression members, are sufficiently succinct to need no explanatory notes. The modulus of elasticity is defined under the heading of Steel for Reinforced Concrete, and its definition will not be repeated here. The modulus of elasticity of concrete varies from one million or less to five or six million. It varies with the intensity of stress; it varies with the kind of aggregate used; it varies with the amount of water used in mixing; it varies with the atmospheric condition during setting. An average value may be 100 per cent. too great or 50 per cent. too small. Added to the absolute uncertainty of its value as determined in plain concrete tests shrinkage of the concrete vitiates assumptions as to relative extensions of concrete and steel jointly stressed. The workability of formulas is immensely hampered by introduction of the modulus of elasticity. If this brought in any needed factor or tended to accuracy, it might be justifiable. It does neither. It is merely a useless refinement of absolutely no value from any standpoint whatever.

The adhesion of concrete to steel is generally stated as *one of the* useful properties in the combination of concrete and steel. Concrete plastered on flat surfaces of steel

will adhere only indifferently. When a steel rod is embedded in concrete, and the concrete is allowed to set in the air, the shrinkage which results causes the concrete to grip the steel rod, and the combination of adhesion and friction due to the gripping makes the rod hold in the concrete. The force necessary to pull a rod out of concrete of 1 :2 :4 mixture, properly made, will be found to average about 500 lbs. per sq. in. of the surface of the rod embedded. This is the unit found on ordinary commercial steel. On such surfaces as that of cold rolled steel the adhesion is not so great, but cold rolled steel has no place in reinforced concrete construction. The surface of hot rolled steel is sufficiently rough to give the above value in adhesion. In the case of a unit such as this a liberal factor of safety should be employed, because of the fact that imperfect concrete or poor work will greatly diminish the available strength. A safe unit for this adhesion is 50 lbs. per sq. in. Round or square rods embedded 50 diameters in concrete at this unit would require a force of 10,000 lbs. per sq. in. to strain the adhesion to 50 lbs. per sq. in. Fifty diameters is therefore a proper depth to anchor a plain round or square rod beyond the point where its full stress occurs.

The coefficient of expansion of concrete, for temperature variations, does not differ much from that of steel. Its value ranges between .0000055 and .0000065 per degree F., while that of steel is about constant at the higher value. The approximate agreement between the coefficient in steel and concrete is of great value in reinforced concrete construction. If there were a large difference, change in temperature would disrupt a concrete member of a structure in which steel were embedded.

Concrete has the useful property of being probably the best preservative of steel known. Even steel that is somewhat rusted will, after being buried in concrete for some time, often be found to be freed of its rust. It has been *thoroughly established* that steel buried in good wet concrete *will be permanently* preserved. To insure the pre-

servation of the steel a wet concrete is needed and one that has a liberal proportion of cement. Dry, rammed concrete is porous and not a suitable constituent of reinforced concrete construction. It cannot be expected to preserve embedded steel.

Another valuable quality of concrete is the capacity to resist fire and not only to retain its integrity and the greater part of its strength through fire, but also to act as protection to steel work. There is perhaps no better fire protection commercially available for steel or cast iron columns than a few inches of cinder concrete, preferably held together with rods or a mesh of steel of some sort. Not all concretes have the same fireproof qualities. Cinder concrete is one of the best, if the cinders be completely burned, for the reason that the aggregate itself being unaffected by heat and the sand being practically so, there remains only the cement that can be attacked by the fire. Also, on account of the cinder concrete being porous, its conductivity is low. The concretes that are best for fire resisting are those in which the aggregates are least affected by heat. Sand and gravel are among the best of the hard stones. Broken brick would make a good fireproof concrete. Sandstone and trap are also good. Granite is not so good, though with the granite broken in small pieces, granite concrete would stand fire better than solid granite. Limestone and marble are not good fire resistants on account of the fact that the heat calcines them, turning them into quicklime. Under the heat of a fire solid stone cracks and spalls off in large chunks. Concrete, and especially reinforced concrete, acts quite differently. The sharp corners will spall off for a small depth, but the broad surfaces will be affected by a calcination acting from the surface in. The depth to which this calcination acts will depend upon the time the fire lasts. It is a slow process and is rendered more so by the lessened conductivity of the calcined stone. A long continued fire may only affect the concrete an inch or so. The driving off of the water chemically combined with the cement absorbs a larg

amount of the heat of a fire. The water does not separate from the cement until a temperature of about 500 to 700 degrees F. is reached, and it requires a much higher temperature than this to complete the dehydration. Concrete conducts heat so slowly that a column of ordinary size would have to be in a hot fire for many hours in order to have the high temperature of the fire penetrate to its center.

Concrete may be used for flue linings where the temperature does not exceed about 600 deg. F. Neat cement tests made at the Watertown Arsenal in 1902 did not show any decrease in strength up to a temperature of 600 deg. F. Small specimens of concrete heated in an oven until the heat penetrates the interior begin to lose strength at about 600 deg. F. for trap concrete and at about 500 deg. F. or less for limestone concrete.

In Engineering World, Jan. 4, 1907, some tests are recorded that purport to show the benefit of tile as a fire protection and the fallacy of depending on concrete for the purpose. As these tests have been given wide publicity, it is pertinent to examine them and weigh their value. Leaving out unimportant details and giving results in close approximations, the tests were as follows. A column about 10 in. square, of limestone concrete, at the age of 23 months stood a crushing load of about 3500 lbs. per sq. in. Another column, identically made, and surrounded with 3 in. of solid porous tile, laid in cement, with metal fabric in the horizontal joints, was subjected, at the same age to a heat of 1500 deg. F. for three hours. Without applying any water to cool the column, presumably, as no mention is made of applying water, it was tested the next day and stood substantially the load carried by the column that was not subjected to the fire.

Another column, identically made, was subjected to the same heat test as the second, without protection, and at the same age. After the fire test water was applied. This *column failed under about* 700 lbs. per sq. in.

The points about these tests that deserve mention are these:

(a) The columns were of limestone concrete, a material acknowledged to be poor as a fire resistant.

(b) The tile used for protection was solid and not, as very generally used, hollow.

(c) The tile was reinforced in the joints with steel mesh, a very unusual and expensive construction.

(d) The tile covered column was not, apparently deluged with water to test its integrity, while the concrete column was so treated.

(e) One hundred by 700 equals 20 by 3500; hence, if we consider that a shell of concrete was totally destroyed by calcination in protecting the inner core, an area of 20 square inches remained good for the full unit load. This means that a thickness of 2¾ in. of concrete was needed to protect the inner core, as against 3 in. of tile. To increase the dimensions of the concrete column by 3 in. each way would be very much cheaper than to add 3 in. of tile of any sort, and the result is a column that is vastly more rigid than would be the tile-covered column.

(f) If we make allowance for the slenderness of the core remaining sound in the bare column subjected to fire, the comparison in (e) would be still more favorable to the concrete as a protection; since the slender column would not be expected to stand as high a unit stress as the original column.

If one of these columns had been made 16 in. square, and had been tested on the same basis as the tile-protected one, it would no doubt have shown that much less than a 3 in. thickness, even of limestone concrete, would serve to preserve the core. This is on account of the fact that the larger core would retain for a longer time a low temperature, not destructive to the concrete, and would take up more of the heat that would otherwise be conducted to the interior.

Concrete is superior to tile as a fire protection. Tile, on *account of* excessive expansion under high heat, will break

up, when in long elements, as surrounding columns or in floor arches or in partitions. This was strikingly exhibited in the Baltimore and San Francisco fires. In hollow blocks of tile or concrete, on account of the fact that the thin shell becomes heated in a short time, the expansion will tend to crack this outer shell off. It is a nice theory that an air space will act as an insulator to keep excessive heat from the vital structural parts, and it would hold true for moderate heat. A hot fire, however, or fire and the subsequent application of water, demands something better than would be required for insulation against moderate heat. In the Baltimore and San Francisco fires tile arches suffered very greatly from the breaking off of the soffit or the under part of the arch. Reinforced concrete, if it be rationally proportioned, that is, with the beams as rectangular beams and not T beams, as advocated in this book, will have ample protection of the steel by the concrete in the lower part of the rectangle, where the heat of the fire is most destructive. The amount of the protection will be in agreement with the size, and hence with the importance in the structure, of the steel protected.

It is more economical to add an inch or two of concrete in the beam or column than to fasten tile to concrete. In any event the latter is easily broken off by expansion, exposing the thinly covered steel. It is immeasurably better to add concrete, even if it were not absolutely needed from the standpoint of strength, for the fire protection it affords, than to load a structure with material that has no other use than fire protection. Such foreign material is an extra load on the structure with no compensation in the way of added strength, whereas a little extra thickness in the concrete adds greatly to the strength and rigidity and longevity of the structure.

It is true of reinforced concrete beams in a greater measure than of columns that concrete added for fire protection is cheaper and better than tile, because the rigidity *of the beams is increased* in a greater proportion by a small *increase in thickness* than would be a column. Also it is

more difficult to make tile hold on the under side of a beam than to make it stand up against a column. Hanging tile ceilings have proven, in recent fires to be specially subject to the destructive action of the heat.

Columns need protection in greater amount near the ceiling of a room where the temperature is the highest. The practice of making knee braces or brackets to the girders, as often followed, is an excellent one; as these serve the double purpose of laterally bracing a building and of adding concrete about the head of a column, where it is most needed for protection. Ornamental caps, molded in the concrete, also serve this purpose.

Reinforced concrete partitions are ideal as barriers against the progress of fire. They will confine a fire in a room as the sides of a stove confine the fire within, provided, of course, that the fire does not go through door openings.

There are systems of floor construction that make use of hollow tile in a rational way. In one of these rows of tile are laid on flat forms at the ceiling level, and between the rows ribs or beams are formed in reinforced concrete, the tile serving to fill the space between the ribs and to present a flat surface for the ceiling plastering. In such construction the destruction of the tile would be of little consequence.

Another system makes use of tile to fill in part or all of the space between a bottom layer of rich concrete an inch or two thick, reinforced with wire mesh and steel rods, and a top layer, two or three inches thick of plain concrete. Sometimes the tiles are separated and concrete poured between them to form ribs, and other times the tiles are laid close. The tiles in this construction carry the horizontal shear, and the concrete above them takes the compression. Being completely enclosed the hollow tiles are protected from fire by the concrete with its embedded wire mesh and rods.

A variation in the above described floor construction *omits* the top layer of concrete. Such a "floor" might be

suitable for a sloping roof, where the live load is not expected to be realized. It is lacking in a prime essential of substantial construction in that the top flange consists solely of a thin sheet of tile, made up of a great number of pieces indifferently joined together by a little mortar used in laying the tile. It would require an inspector watching the laying of every tile to insure the filling of the joints with mortar. Flushing would not fill these joints, as the liquid mortar would be wasted by running into the hollows of the tile, if flushing were attempted. The difficulty in securing filled joints lies in the fact that the joints that count are where the thin edges of the tile meet. These remarks apply also to ordinary tile arches, though not with equal force, since the thrust in an ordinary tile arch does not approach in intensity the compression in a system of hollow tile, say 5½" deep with ¾" "metal" having 1" of concrete or mortar underneath, with its embedded steel, on a span of sixteen feet. These are the dimensions of a floor measured by the author.

The ability of concrete to resist the passage of water though it varies all the way from that of a sieve to that of a good cork. The way to make concrete that will allow the flow of water through it with little impediment is to mix it dry, that is, with a minimum amount of water, and then to ram it or not to ram it in place, as the ramming has little to do with it. Ramming will bring the stones together and help the adhesion; it will not get rid of the general porosity. The only way to make concrete of itself waterproof is to mix it very wet. Two or three inches of wet concrete will hold water back better than ten or twenty feet of dry rammed concrete. This would appear quite extravagant, if it were not a fact proven true by experience. The fact that by using a wet mixture in the thin walls of a tank water can be completely retained is sufficient refutation of the charge so often made that concrete is inherently porous.

If a little cement mortar be mixed in a tumbler, the water being slowly added, it will be seen that while the

mixture is dry, or the consistency of "moist earth," there will be bubbles in it. Ramming or compacting the mortar will not serve to eliminate these bubbles but may break them up. The air is imprisoned and has not the buoyancy to force its way to the surface. Some of it may be forced out on account of the porosity of the mortar, but the ultimate result is only a reduction of the magnitude of the pores. When more water is added, the mortar assumes a liquid consistency, and the air bubbles will rise to the surface. It is urged against liquid concrete that the excess of water above that which the cement absorbs in the hardening process, when it evaporates, will leave voids in the concrete. By this reasoning it is concluded that wet concrete will be porous, and by reversing it, it is contended that the proper consistency for the maximum watertightness is that in which there is just enough water for the needs of the cement in hardening. Reasoning like this would make a cork a very poor thing to keep back water. Cork is quite porous. It can be made to absorb hot water, and it will give it off again in drying. Its substance can be compressed to a fraction of its normal volume. And yet it is practically perfectly water tight, while other woods or barks, weighing much more and apparently much more dense, do not possess this property. The best way to keep water out of concrete is to put lots of water in it in the manufacture. A rich concrete is necessary, for water tightness, and a mixture that will compact well. Small gravel is good for this purpose because of the density of the stones themselves and because gravel has the property of packing well on account of the comparatively small friction between the stones. Thorough mixing is another necessity, to incorporate the cement and water and to distribute it uniformly throughout the mass.

Some clay in the sand in a finely divided state or added to the mixture, if it can be thoroughly mixed through the mass, or clay in a semi-liquid state has been known for some time to add to the density and watertightness of concrete. In Eng. News, Sept. 26, 1907 Mr. Richard H. Gaines

describes some tests in which 5 to 10% of the cement of mortar was replaced with an equal quantity of dried and finely ground colloidal clay intimately mixed with the cement. The result was a watertight product much stronger than the cement mortar.

Some concretes become more impermeable with age. Concrete which when new may not be quite watertight, may become so by the action of water passing through it. This may be due partially to uncombined silicates being dissolved by the water and re-deposited, or to the swelling of the fine cement particles in the presence of water. Possibly also solid matter in suspense in the water clogs up the interstices as a filter becomes clogged.

It is true that even with wet concrete pockets are liable to form, and poor batches may find their way into a wall or tank. These can be stopped up or remedied by rich mortar filling all of the pores. In a cistern built under the author's supervision perfect watertightness was attained by the use of wet concrete and careful plastering of all holes with neat mortar while applying a wash of neat mortar with a brush, after thoroughly wetting the surface. The neat cement wash exposed at once any large pores in the concrete, and these were plastered up by troweling. This wash was made the consistency of thick cream. After 24 hours the surface was again washed with the cement cream, and a third coat was applied after another interval of 24 hours. In addition to this treatment the contractor washed the surface with a mixture of hydrated lime and soft soap. The author is of the opinion that this latter did not affect the watertightness, as it appeared not to be permanent. (See Eng. News, Sept. 28, 1905, p. 330).

That concrete of itself can be made practically watertight is proven by numerous tanks and sewer pipes and some water pipes that have been made of concrete untreated and not "waterproofed." Tests of mortar and concrete have demonstrated their ability to withstand water *under pressure. In Trans.* Assoc. of C. E., Cornell University, *Vol. XIII, Mr.* A. B. Moncrieff describes tests or

blocks of concrete, 2 ft. each way, in which a water pipe was embedded, the pipe being surrounded by a 5 in. bulb of hemp and small rope. Some of these test blocks allowed only 1-50 of a pint of water to pass through in two weeks under 100 ft. head of water. At the end of 80 weeks the tests were repeated with a head of 200 ft. The results were about the same.

In Engineering News, June 26, 1902, p. 517, Messrs, J. B. McIntyre and A. L. True describe some tests on gravel concrete 5 in. thick that had set 24 hrs. in air under a damp cloth and a month in water. They found that concrete specimens containing 1:1 mortar, and in which the mortar was 30 to 45% of the whole mass, were watertight. These stood pressures as high as 80 lbs. per sq. in. for 24 hrs. without leak. Some of the concrete specimens having 1:2 mortar and 40 to 45% of mortar in the mixture were also impermeable, as well as the 1:2:4 and 1:2.5:4 mixtures.

In Engineering Record Dec. 14, 1907, p. 661, some tests are given by R. T. Surtees, C. E. Newton-le-Willows, Lancashire, England, that show remarkable results. Some plugs of concrete 5 in. thick were made in pipes and water under pressure was brought to bear against them. Two of these stood a pressure of 35 to 50 lbs. per sq. in. for thirty days without leakage. In a repetition of the tests one specimen was allowed to set where exposed to the sun most of the day, another was allowed to set in a shaded place, and two others were allowed two days under a damp cloth and 28 days immersed in water. The concrete of the first shrunk and did not fill the pipe. The second showed a very slight dampness, which soon took up. The other two were perfectly watertight. The pressure on these tests was increased to 260 lbs. per sq. in. With pressure varying between 50 and 260 lbs. for two days no sign of dampness appeared on the outside. One of the blocks was cut out to see how far the water had penetrated. It was seen to be traceable for a distance of only 1⅛ in. The concrete of these tests was made of two mixtures, namely, 1 part of cement, 1 part of sand, 1 part of crushed gravel, 1 part of

gravel screened to ¾ in., and a mixture of 1:1:1½:1½ of the same ingredients respectively. The same experimenter made a reinforced concrete pipe and subjected it to 120 lbs. No sign of dampness appeared on the outside of the pipe.

Troweling increases the impermeability of concrete. Troweling should be done in such positions as the top surface of an arch and the exposed surface of a sea wall.

Concrete has been found to be suitable for tanks in which to freeze water in the manufacture of ice. At the annual meeting of the Am. Soc. of Refrigerating Engineers, at New York, in 1907, Mr. Abram Day describes some ice tanks of reinforced concrete that have been used for some years with satisfaction. In some tanks Mr. Day used 1:1:3 concrete of trap rock and in others 1½:1½:2½. He finds the best size of aggregate to be stone about ⅜ in. to ¼ in. The inside of his tanks are either left rough or washed with neat cement. No waterproofing is used. [See Eng. News, Dec. 12, 1907, and Ice and Refrigeration, Dec. 1907.]

Another useful property of concrete was brought out at the meeting referred to in the previous paragraph. Mr. John E. Starr described a nine-story cold-storage warehouse built entirely of reinforced concrete, with a double facing wall filled in between with cork. Additional insulation was also provided by coating the tops of all floors with cork boards, two to four inches thick. It is a remarkable fact that though the inside of this house is frequently at as low a temperature as five below zero, F., with the outside wall above 100 deg. F., no signs of cracking, due to expansion, have ever been noticed.

Hollow blocks of concrete afford excellent insulation against moderate heat or to retain heat in a building.

The ability of concrete to stand weather and atmospheric conditions is often much better than that of the stone composing the aggregate of the concrete. A poor stone is improved by being made into concrete and surrounded and protected by a mortar of sand and cement. Hard stone such as *granite and trap* are not as durable nor as strong, *made into concrete*, as the original stone. These stones

however, make the strongest kind of concrete, and the use of trap, especially, in concrete has the advantage of being a means of utilizing this stone for building purposes, when it would be scarcely possible to use it at all as a building stone. Trap is very refractory under the dressing tool and can only be employed in rough blocks. Some stones that would develop planes of fracture in a structure made of block stones have these planes fractured in the crusher. They are therefore rendered safe against failure by cracking. Stones that by themselves suffer surface disintegration would be subject to the same action, if exposed on the surface, though the cement and sand of the concrete will protect them very materially. If the mortar of the concrete be worked to the surface, stone that would suffer surface disintegration may be quite durable in concrete. Slag cement should not be used for concrete that will be dry, if used at all, it should be where the concrete is always wet. Good Portland cement concrete will be durable either wet or dry.

One of the most trying situations for concrete is sea water between high and low tides. Much has been written on the subject of the chemical effect of the salts of sea water on cement and concrete. That the deterioration is a surface action almost entirely and that it is generally confined to the zone between high and low tide levels suggests that the action is at least partially mechanical. The formation of salt crystals in the pores of the concrete, by the evaporation of the salt water, by their swelling action may have something to do with this surface disintegration.

It is a fact, and one that the author has found to be not generally known by practical chemists, that silicates, when ground to a fine powder, glass, for example, are soluble in acids and alkalies that do not affect the solid lumps. Ground glass is even soluble in pure water. Crushed rock will make soil in a very short time, whereas the same rock, *in large pieces,* may withstand the weather a long time *before disintegrating.* Water acts on the ground rock in

a very short time. It is said that holes can be blasted in the rock in certain parts of the Florida Keys and crops planted in the broken stone. Cement clinker is a sort of glass. When ground to a fine powder, it is partially soluble in water. This can be seen by stirring cement in a large quantity of water, preferably hot. A glass like scum forms on top of the water after it stands a while. If allowed to absorb water and to harden without being subject to conditions that take away the water, cement will unite with the water and form a permanent compound. If the amount of water is stinted, as in the "moist earth" concrete mixtures so popular with many, it is impossible for all of the grains of cement to find sufficient water to make them into a stable compound. They remain, therefore, in some degree, in the state of ground silicates, easily dissolved by water. With Portland Cement, sloppy concrete and a smooth finish on the surface exposed to sea water, or any wave action, is the best precaution against dissolving action.

The continuous washing of the surface of concrete by waves of water; either salt or fresh, would be expected to carry away any uncombined silicates in the cement; whereas, if the water were still, the dissolved silicates would not be carried away, but, as is possibly the case in still water, they would be redeposited in the pores of the concrete. It is probably this dissolving that is responsible for the pitting and roughening of cement pavements caused by rain. While this is a chemical action, the mechanical action of the beating water supplies the destructive element. Dry concrete and excessive troweling to give a glossy surface are very commonly resorted to by pavers. It is not uncommon to see the same pavements pitted after some time of exposure to weather.

In sewers and water pipes this is not exhibited in the same way, because mud deposits in the pores and protects the concrete where the scour would be the most.

Concrete subject to the action of sea waves should have *a coat of rich mortar* plastered very smoothly. Facing con-

crete with granite between high and low tide levels is very often practiced. This is a sure way of rendering a sea wall permanent.

There is a cement called iron-ore cement, manufactured in Germany, which is said to resist the action of sea water. This cement is made of iron-ore and limestone instead of clay and limestone.

The weight of stone concrete generally runs about 150 lbs. per cu. ft. Heavy stones, such as granite and limestone may exceed this. Sandstone concrete may weigh less. Concrete mixed with a minimum amount of water will weigh less than medium or wet concrete, a fact which testifies to the porosity of the former. The weight of sandstone is about 150 lbs. per cu. ft.; of granite, 170; of limestone, 170.

Cinder concrete weighs about 110 lbs. per cu. ft. when dry. Wet cinder concrete should be calculated to weigh about 125 lbs. per cu. ft. in designing the forms.

Notes on General Design and Construction

Under this heading will be given some notes bearing on the general design and treatment of structures that do not come under the head of any of the other general subjects treated.

Drainage. Retaining walls and arches should be well drained by a system carefully planned to perform its work. Iron or steel pipes should be avoided, if the water passing through them runs over the exposed face of the wall; because the rust will disfigure the wall. A collapsible wooden core, say of a round or square piece split longitudinally on a slant, would allow the removal of the pieces separately. Or a box of thin wood could be used and broken up on removal. Or a smooth hard wood core could be used and drawn out before the concrete has a hard set. Clay tiles placed in concrete make good permanent drains. In

building a wall, there should be a lot of loose stones placed where water would collect and where the drain is located, so that it will not clog up. It would not be out of place to have a drainage system in a building. In a fire damage by water is often greater than by the fire itself. A fire in the upper story of a building may be extinguished by the application of water, which, if it is not carried away, may run down the stairs to floors below and work damage there.

PROVISION FOR EXPANSION AND CONTRACTION. Calculations for temperature stresses in concrete are of little if any use. Shrinkage in concrete due to setting in the air is a more important factor in changing its size than temperature variation. If this were elimnated, calculations using temperature changes would have some meaning. However the actual change in temperature, as in an arch, would not be very much, especially if the arch is one having fill over it. Concrete and earth are both poor conductors of heat, and the range of temperature in the body of the arch would not approach that which could be expected to take place in a steel structure. It is usually in arches that temperature stresses are the most minutely elaborated. There is undoubtedly more or less uncertainty in the stresses in an arch due to temperature changes, but the same is true in a larger measure of the effect of shrinkage. Uncertainties should be covered by a factor of safety or by liberal design: it does not minimize them to attempt to cover them by fictitious though elaborately calculated stresses. Steel reinforcement in all parts of an arch or other structure is one of the best safeguards against shrinkage and temperature stresses. To build a structure so that shrinkage may take place during the building is another. In an arch it would be better, where possible to place the entire arch ring, leaving a groove and steps for the spandrel wall to be poured after the forms of the arch ring are removed. In a building walls or partitions would be better to be separate from the columns, let into recesses in the latter and placed after the columns have set. In a long

structure two sets of columns and girders at a dividing plane would give opportunity for shrinkage to act. Provision for shrinkage is only a makeshift, and a useless one, unless carried down to the foundation.

ARCHES IN SERIES. In a long line of arch spans it is legitimate to make the piers between the adjacent spans heavy enough to be rigid against live load only on any span, if the arches are equal and the dead load thrust balanced. However, there should be an occasional pier that will take the full thrust of one span, so that if one span should fail, the entire system will not come down. Every third or fifth pier may be an abutment. Forms should not be removed in a long line of arches in a span adjacent to a pier that is not capable of acting as an abutment.

BOX CULVERTS. Flat slab tops on box culverts are preferable to arches for several reasons. They do not require abutments but only supporting side walls. When fill is being made between wooden trestle, a flat top box culvert can be introduced between trestle bents, where often an arch culvert and its abutments would require the removal of one or two bents.

TANKS. There is not much economy, if any in using a reinforced concrete tank as compared with a steel tank. The shell of a steel tank can be stressed to 20,000 lbs. per sq. in. or more, and the tank will be safe and tight. Many oil tanks are designed with a unit stress, on the vertical section, of about 24,000 lbs. per sq. in. Steel em-
bedd e and stressed to these amounts would
r and of course the cracks would destroy
k as a receptacle for liquids. There is
n cost per pound of steel rods placed
ete tank and the steel shell in place
is therefore a large margin in favor
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re they would be more economical. The
ality of reinforced concrete makes it more
y cases than steel. Tanks with small ten-

sion in the shell may require to be made of steel plates much thicker than the mere tension would demand, so that small tanks might be as cheap in reinforced concrete as in steel.

CISTERNS. A cistern with an approximately flat bottom would often be better to have the bottom somewhat concave. Shrinkage cracks are less liable to occur in a curved bottom, and with the bottom concave the cistern can be emptied and cleaned better than with a flat bottom.

SMALL COLUMN PEDESTALS. In making small column pedestals it is well to make as few different sizes in a given structure as practicable, at the expense of using more concrete than is required in some of them. It simplifies the form work to make duplicate sizes. Steps in the sides are preferable to a batter, especially if the top is small and the batter large. It is hard to pour concrete in the small hole and hard to tamp it through the same.

STAIR SUPPORTS. Slabs and beams for the support of stairs are to be calculated for a span equal to the horizontal projection and not the actual length, though of course the weight of the parts are found from their actual dimensions. When triangular blocks are cast on a slab for the steps, these should not be counted in any part of the depth of slab. The slab should be proportioned for a depth exclusive of these blocks. At angles, as where a landing joins the slope of the stairs, there should not be a sharp corner in the slab: there should be a thickening of the slab, and reinforcing rods should be given gentle curves, say with a radius 50 to 80 times the diameter of rod.

BEAMS AND GIRDERS. As few sizes as possible of beams and girders should be used, and their spacing should be as regular as practicable. For economy, the deeper the beam the less it will weigh, just as in wooden construction and, generally, in steel work. However, deep beams loaded to their capacity are apt to be weak in shear. Also deep beams *may be too narrow* for other reasons. They w *further, encroach on* clearance, or they may necessitate

ditional height to a building. Girders and beams should be placed at columns rather than near the columns so that they will help to stiffen the building.

SHAFT HANGERS. Shaft hangers may be attached to beams by means of bolts placed in the concrete having the threaded end projecting to receive the hanger; or threaded castings may be built in the beam, either in the side or bottom, to these the hanger may be attached, or timbers or steel angles may be bolted on; or pieces of gaspipe may be left through the beams near the bottom and bolts run through them to which timbers, or blocks or longitudinal angles may be bolted; or clamps may be used on the beams, held on by friction.

HOLES FOR PIPES AND WIRES. Holes should be left in floors in the building of them for all pipes and wires that may be placed. It would be better to leave too many than not to leave any or not to leave enough. It is easy to cement up such holes but very difficult to cut them in hardened concrete.

BATTERED ABUTMENTS. It is unnecessary to give abutments a batter on the front face. It appears to be a rudiment of some popular notion of stability that a small batter, say of 1 to 12, will add rigidity to abutments, wing walls, sides of culverts (inside), etc. It does not pay for the trouble.

VERTICAL PIPES. If possible vertical pipes should be located where they will not run between a column and its fire protection. Expansion due to heat will cause the pipes to buckle and will tend to pry off the fire protection. This will apply to any building where the fire protection is separate from the structural column, as in cast iron or steel column construction.

HEAVY GIRDERS. In general, reinforced concrete is neither appropriate nor economical for very heavy girders, either long span girders or short ones taking heavy concentrations. Steel girders are more suitable in such cases, *very long* spans the dead weight of the girder is ex-*e: in short* spans carrying heavy concentrations the

embedment of the steel is apt to be insufficient for the stress, also shearing stresses are difficult to provide for. Shallow beams are best made in steel also, as concrete beams shallow in depth are clumsy on account of having to be so wide.

SLENDER COLUMNS. Slender columns should not be attempted in concrete. They are better to be of cast iron or of steel, protected with concrete if necessary.

REINFORCING WALLS AT CORNERS. Walls that meet at an angle should be tied together at the intersection to prevent cracking at the corner.

SCHEME FOR FLOOR SUPPORT. A very simple and efficient reinforced concrete floor for either a highway or a railroad bridge can be made by running rods transversely from girder to girder to act as bottom reinforcing rods in the floor slab. The rods should be plain, round, threaded on the ends, and should pass through the steel work, holes being left for the purpose. If the slab rests on a shelf on the side of a beam or girder, the holes may be in the web just above the shelf. If it is a through span, the slab resting on the bottom flange of the girder, some of the rivet holes, say at intervals of 6 or 9 ins., may be left open. In a deck span the web plate may be extended to take the holes for reinforcing rods. This slab construction is admirable for highway bridges. The entire roadway may be one continuous slab from truss to truss or girder to girder, passing over the stringers, with the same rods to act as reinforcement. Over this slab brick or block or asphalt pavement may be laid. The lateral system on a bridge with a floor of this sort need only be strong enough to take care of lateral forces during erection, as the concrete slab will give ample stiffness.

SURFACE FINISH. Every contract for concrete work should include a clause requiring that the exposed surface be properly finished. The finish should be appropriate to the nature of the work. In some cases this may mean *merely a washing down* to remove dirt and efflorescence *after removal of forms*. An otherwise good job of con-

creting may appear very poor on account of failure to treat the surface properly, and the owner may have to go to considerable expense to improve the appearance of a structure by reason of failure to specify the kind of finish desired.

WATERPROOF PLASTER. A mixture, by volume, of 3 parts of litharge, 1 part of glycerine, 48 parts of Portland cement, and 48 parts of sand makes a strong, adherent, and dense plaster, which will repel water. Hot paraffine is sometimes ironed into concrete to waterproof the surface. Linseed oil, painted on until the concrete will absorb no more, is also used to waterproof concrete that has not been properly made and lacks density.

WHITEWASH. Whitewash for concrete surfaces should be durable and adhesive. The following is recommended. Slake with warm water, half a bushel of lime, covering it during the process to keep in the steam; strain the liquid through a fine sieve or strainer; add a peck of salt, previously well dissolved in warm water, 3 lb. of ground rice, boiled to a thin paste and stirred in boiling hot water, ½ lb. of powdered Spanish whiting, and a pound of glue which has been previously dissolved over a slow fire; add five gallons of hot water to the mixture. Stir well and let it stand for a few days, covered from the dirt. Strain carefully and apply with a brush or a spray pump. It should be put on hot. Coloring matter may be put in to make various shades.

SOME USES OF CONCRETE. Some of the more uncommon situations in which concrete can be used to advantage are the following:

For props in coal mines, in place of timber.

To line steel coal or ash hoppers or bins.

For fence posts. These are usually made square and tapered and have a reinforcing steel wire near each corner, these tied together with wires.

For roof tiles or shingles. These may be reinforced with a wire mesh or metal lath of some kind.

To protect wooden piles in sea water from toredo. To

accomplish this the concrete may be molded into a cylinder around the pile. Steel reinforcement must be used to hold the concrete from breaking off. Another method is to use concrete half-cylinders surrounding the pile, or sewer pipe of clay or concrete if they can be threaded over the top of the pile. The space between the pile and the surrounding cylinder is then filled with sand. In the event of a break in the surrounding cylinder the sand will run out and the damage can be detected and repaired.

Estimating Cost.

The cost of a structure is a function of the number of pounds of steel or cast iron, the number of cubic feet or yards of masonry, the number of bricks, the number of feet of board measure, the number of square feet of paving, of lineal feet of handrailing, etc., that go to make up the whole. An estimate of the cost requires careful calculating of all of these as well as a knowledge of a fair unit price at which they can be put into the structure.

Cost estimates must be based on unit values. The accuracy of the estimate will often depend upon the particular unit at which the estimate starts. Sometimes greater accuracy will result from tracing back the unit cost of the raw materials of manufacture: often such tracing is productive of only confusion with no increased accuracy. For example, there is nothing gained by analyzing the cost of a pound of steel or a barrel of cement. These materials have certain prices fixed by those who sell them. In the matter of working up materials there are also arbitrary elements. The cost per unit will depend upon the particular plant or equipment employed and its fitness to handle the work most economically.

The plant or equipment needed to do a piece of work should be selected with a view of the size of the work and the time in which it is to be finished. Large equipment *cannot, in general,* be used economically on a small job, *and small equipment* cannot be used economically on a

large job. The size of a concrete plant should be such that its normal daily capacity is about equal to the amount of concrete that it is desired to turn out per day. For maximum economy a plant should be employed continously. If stops must be made to wait for forms to be put in readiness, or for other causes, the concrete will cost more than if the work of the concrete mixing can be carried on continuously.

For small concrete jobs, such as pavement work, hand mixing is more economical. Small batches may be mixed with a hoe or shovels in a box. Half-yard batches should be mixed on a platform by at least two men with shovels. The platform may be made of a steel plate or of boards placed with close joints on a frame.

A typical gang mixing and laying one-half cubic yard batches is the following: 1 foreman, 2 men delivering sand and stone, 1 man delivering cement, 2 men mixing 2 men delivering concrete, 1 man tamping. At $3 per day for the foreman and $1.50 per day for each of the other men the cost per day of this gang is $15. The gang should turn out about 20 to 25 cu. yds. per day. This is a cost of 75 cts. to 60 cts. per cu. yd. for labor.

A typical gang for mixing and laying by hand cubic-yard batches is as follows: 1 foreman, 3 men delivering sand and stone, 1 man delivering cement, 4 men mixing, 3 men delivering concrete, 2 men tamping. The cost of this gang at the same wages as above is $22.50 per day. They should turn out about 40 cubic yards per day, making the cost of labor 56 cts. per cu. yd.

The above examples give about average conditions and show the cost of labor on hand mixed concrete in heavy work where mixing and laying can go on continuously. If labor is cheap (and efficient) the unit cost may be less, and vice versa. If materials can be deposited for easy handling, as when they are laid close to the mixing board and need only to be measured the unit cost will be reduced *accordingly*, whereas long hauls or high lifts, either before *or after mixing* will add to the cost very materially. It

the gang cannot be continuously employed, costs may be two or three times as much as the above. Concrete deposited in narrow forms will also cost more per cu. yd. than in massive work.

With mechanical mixers the cost of mixing concrete will be less than by hand mixing, though the extra cost of skilled workers to run the engine and mixer helps to balance the costs. Batch mixers should turn out about 20 batches per hour.

Current prices for which similar work is being done in localities situated about the same distance from the source of supply afford a sound basis upon which to gage the cost of work. It is best for the engineer not engaged in the estimating of cost to the contractor or manufacturer to use as a base the unit cost of work in place, rather than to analyze the elements that go to make up that cost, such as material, labor, freight, hauling, profit, etc. The contractor's profit is an elastic factor, depending upon the size of the work, the risk, and many other considerations. The cost of manufacture is variable. Some shops can make heavy work cheaper than others, while others can handle light work more economically. It is not the purpose here to analyze the cost in shops and mills, so much as to give more general data for determining the probable cost of ordinary building and bridge work, as well as to point out some of the special cases where costs are apt to be more or less than the average. Average costs will prevail near the railroads and within radii of 50 or 100 miles of the commercial centers. Freight rates average about ¾ to one and one-half cents per ton-mile. Long pieces requiring several cars and not weighing enough to load them to their normal capacity will cost more per ton than materials that can be shipped in full car loads. Partial carloads are charged at a minimum car load rate, say one-half of the capacity of the car. Where more than one car is required, one car is charged at this minimum rate and *each other car at one-half* of this amount, if the actual *weight of the material* shipped is not over that total.

Hauling under ordinary conditions costs about 50 cents per ton for structural material.

The actual cost of hauling crushed stone 1½ miles in some macadam paving was found to be 26.6 cts. per ton when drawn from the crusher bins and 31 cts. per ton when drawn from the piles. The contract price was 32½ cts. per ton.

The hire of a dumping wagon and team and driver is about 4 dollars per day; that of a horse and cart and driver is about three dollars per day.

The actual cost, with stone free at the quarry, of laying macadam pavement (5″ layer large sized stone, rolled; 2½″ to 3″ layer of medium sized stone, sprinkled and rolled; about ¾″ of fine screenings, sprinkled and rolled) was 42 cents per square yard. The average weight of stone was .3 tons per sq. yd. (Eng. News, Oct. 8, 1903).

The actual cost of quarrying and crushing stone in the above mentioned work was 42 cts. per ton; in which coal delivered cost 4 dollars per ton, a driller $1.75 per day, helper $1.50 per day, engineman $2 per day.

The cost of mixing materials and laying the same in making the Buffalo breakwater was as follows:

Laying Materials	17.4 cts. cu. yd.
Mixing Materials	12.9 cts. cu. yd.
Placing mixed materials	14.6 cts. cu. yd.
Total	44.9 cts. cu. yd.

Sometimes the gravel and sand for a piece of work can be found at or near the site, thus greatly reducing the cost of concrete made of the same.

Bricks may be hauled direct from the works without the expense of loading and unloading on cars.

Extra hazardous work should have something added to the estimated cost to allow for the risk taken by the contractor. Work that must be finished in a short time should have the estimate increased, especially if a penalty attaches for failure to complete by a specified time. If the season is a poor one for the class of work, still more expense is liable to be incurred. Erecting of bridges over streams in

flood time may be attended by serious difficulties and expensive delays.

Large contracts, as a rule, cost less per unit than small ones. The placing and removing of the contractors plant on a job often requires considerable time. If the magnitude of work does not justify bringing labor saving machinery to the site, the extra labor will make the smaller job more expensive. Large orders of materials may be placed at lower rates than small ones.

Where labor is the principal item of cost in any work, less certainty can be expected in the estimate of the cost, and little agreement between prices bid by contractors; whereas materials that are regularly manufactured should vary but little in cost.

There are some general rules that will be found very useful in making a rough estimate of the cost of structures and checking against large errors in more careful estimates. The cost per square foot of area covered by a building having practically one floor will be nearly constant for different sizes of buildings of the same class. Higher buildings will have a cost per cubic foot nearly constant for a given class of building. For ordinary lengths of spans the cost of reinforced concrete bridges per square foot of floor does not vary much.

The cost per square foot of the area covered by buildings of the World's Columbian Exposition for nine of the principal buildings varied between 75 cents and \$2.35, and averaged about \$1.50. The Administration building cost \$9.18 per square foot. The cost per square foot under roof of eleven of the principal buildings of the Louisiana Purchase Exposition varied between 61 cents and \$1.49 with an average of \$1.12. Festival Hall cost \$5.23 per sq. ft. Fine Arts Building (central building) cost \$9.88 per sq. ft., U. S. Gov't. Building cost \$2.31 per sq. ft. The eleven buildings at St. Louis have timber framework. The Chicago buildings had steel frames. The Fine Arts Building at St. Louis is permanent and fire-proof. The U. S.

Gov't. Building has steel arches. The cost of the steel work alone was 65 cents per sq. ft.

The cost per square foot of floor of what is probably the longest stone arch in the world; namely, the bridge at Plauen, Saxony with a span of 295.2 feet, was $4.65. Low cost of labor and availability of stone close to the bridge made its cost much lower than such a bridge could ordinarily be built for (Eng. News, Jan. 28, 1904).

Fern Hollow Bridge at Pittsburg, Pa., cost, including the masonry $4.06 per sq. ft. of floor. This is a plate girder arch with viaduct approaches. The arch span is 195 feet. It was estimated that a stone arch bridge would have cost nearly three times as much. (Eng. News, Feb. 26, 1903).

The cost of a double track stone arch railroad bridge of 64-foot spans at Watertown, Wis. was $4.35 per sq. ft., including removal of old bridge. (See Eng. News, Vol. 49, p. 266).

The cost of the P. R. R. Co.'s four-track stone arch bridge at Rockville, Pa. was $5.03 per square ft.

Following are the approximate costs of reinforced concrete arch bridges per square foot of roadway and sidewalk, end to end of abutments:—

Bridge at Dayton, O., Spans 69 to 88 ft. Pavement of bituminous macadam. Assuming abutments 20 ft. each, cost per sq. ft. = $3.63 (Eng. News, May 19, 1904).

Bridge at Dayton, O., Spans 80 to 110 ft. Cost, including provision for temporary traffic and removal of old bridge, $3.63 per sq. ft. (R. R. Gazette, May 4, 1904).

Bridge at Forest Park, St. Louis, Span 45 ft. Macadam pavement on roadway. Cost per sq. ft. $3.63. (Eng. News, June 11, 1903).

Bridge at Laibach, Austria. Span 108 ft. Cost $32,000 ($4.14 per sq. ft.) The metal work of this bridge cost about $6,000 and the ornamental work $2,000. (Eng. News, *July 16, 1903*).

Bridge at **Brooklyn,** N. Y. Skew arch. Cost $2.85 per

sq. ft. Contains 91,360 lbs, of steel and 1300 cu. yds. of concrete. (Eng. News, Dec. 30, 1903).

Bridge at Waterloo, Iowa. Seven spans each 72 ft. Cost per sq. ft. $1.65. This does not include floors and pavements. Bridge contains 7200 bbls. Portland cement, 5000 cu. yds. crushed rock, 3000 cu. yds. sand, 110 tons of steel ribs, 6,000 cu. yds. of spandrel filling. (Eng. Record, Feb. 13, 1904).

Bridge at Des Moines, Iowa. Spans 100 ft. Spandrel walls and arch ring faced with brick. Cost per sq. ft. $4.48. (Eng. News, May 14, 1903).

Bridge at Topeka, Kan. Longest span 125 ft. Cost $4.51 per sq. ft. (Eng. News, Apr. 2, 1896).

Bridges at Niagara Falls. Cost of two bridges $4.60 per sq. ft. (Eng. Record, Feb. 16, 1901).

Bridges in Porto Rico. One bridge having 1—120-ft. and 2—100-ft. spans and rather high piers cost $8.17 per sq. ft. and one having 3—70-ft. spans cost $5.29 per sq. ft. (Eng. News, Aug. 1, 1901)

Bridge at Washington Street, Dayton, O., about $3.30 per sq. ft. (Eng. Record, Mar. 2, 1907).

Bridge at Sandy Hill, N. Y., $2.43 per sq. ft. (Eng. Record. May 4, 1907.)

Bridge at Jacksonville, $3.00 per sq. ft. (Eng. Record, May 18, 1907).

Bridge at Mishawaka, $3.82 per sq. ft. (Eng. Record, July 7, 1906.)

Bridge at South Bend. $3.32 per sq. ft. (Eng. Record, July 28, 1906.)

Bridge at Philadelphia, $6.85 per sq. ft. (Eng. Record, Nov. 17, 1906.)

Bridge near Goshen, O., $4.67 per sq. ft. (Eng. Record, Mar. 30, 1907.)

Bridge over Rock Creek, Washington, D. C. (Boulder-faced span, illustrated in this book), $5 per sq. ft. (Eng. Record, *Vol. 46, p. 151.*)

The cost in 1903 of a large reinforced concrete factory

building was 6.4 cts. per cu. ft. This was for the building alone, not including plumbing or furnishings.

Mr. H. G. Tyrrell (R. R. Gazette, Vol. 37, No. 18) made comparative estimates of a large factory building designed to carry 100 lbs. per sq. ft. of live load (six stories and basement), and found that a building of heavy wooden interior construction with brick floors, and cast iron columns in the lower two tiers would cost 6.2 cts. per cu. ft. or 83 cts. per sq. ft. of area of floors; the same building with concrete steel floors on a steel frame work would cost 10.2 cts. per cu. ft. or $1.36 per sq. ft. of area of floors. This did not include furnishings or stairs.

The cost of a reinforced concrete power building per cu. ft. above ground was 7.7 cts. (See description, Eng. Record, Apr. 15, 1905, p. 438.) The total cost of this building was $225,000.

The cost of a brick building with slate roof on timber will probably be from 8 to 14 cents per cubic foot of its volume.

The cost of a mill building with sides and roof of corrugated iron will probably be from 75 cents to $1.50 per square foot of plan.

The cost of apartment buildings and department stores as usually constructed will be from 20 to 30 cents per cubic foot of volume.

Office buildings will usually run from 30 to 60 cents per cubic foot.

City dwellings will run from 10 to 30 cents per cubic foot.

Brick veneer dwellings will cost about 8 cents per cubic foot.

Window and door frames as ordinarily made for mill buildings cost about 25 cents per square foot in place estimating the dimensions out to out of frames. Galvanized iron louvres of No. 18 iron cost about the same.

The cost of furnishing clips and rivets and putting up corrugated iron is about $2.00 per square of 100 square feet.

Erection of plain structural work costs 9 to 10 dollars

per ton; of frame work of office buildings 10 to 12 dollars per ton; of mill buildings 11 to 15 dollars per ton. Complicated work of many small parts and light tonnage, such as angles and tees for roof tile, may run as high as 28 to 30 dollars per ton to erect. Bridge truss work will cost 15 to 20 dollars per ton. These figures include furnishing falsework, also the painting.

The painting of structural work costs about one dollar per ton for each coat.

The driving of field rivets costs from 5 cents to 20 cents each, depending upon the accessibility of the rivets and the number of times that scaffolds must be moved in a day. A riveting gang costs about 18 dollars a day. For ordinary work 12 cents per rivet is a good average.

The hire of an engine and derrick is about 30 dollars per week. That of an engine and concrete mixer is about the same. This does not include any men to operate the same.

The hire of a road roller with coal and operator is about 12 dollars per day.

The hire of a work train and crew, coal, etc., is about $22 per day.

The cost of galvanizing structural work is about twenty dollars a ton.

The cost of corrugated steel roofing or siding per square of 100 square feet, at 3½ cents per pound for material and $2.25 per square for erection and painting is about $11.75 for No. 18 and about $9.00 for No. 20. Galvanized roofing, at one cent extra for galvanizing would cost about $14.50 for No. 18 and about $11.00 for No. 20 per square erected.

The cost of concrete-steel roofing on 15-foot spans is about 25 to 30 cents per square foot, exclusive of covering. Concrete-steel floors for ordinary loads on spans 8 to 10 ft. cost about the same. Heavy floors cost 30 to 40 cents per sq. ft. Cement finish on floors costs 7 to 10 cents per sq. ft.

The cost in 1903 of a 54 inch self-supporting steel stack *110 ft. high of 5-16", ¼"*, and 3-16" metal, including lad-

der, painted on outside, with base casting, but not anchor bolts or foundation, was $1,200. Breeching of 3-16" metal, 5 ft. in diameter or oblong and same area cost $9.00 per lineal foot.

The cost in 1902 of a stack 10 ft. inside diameter and 180 ft. high, of porous brick, was $7,375, not including foundation. A 125-ft. x 6-ft. porous brick stack, including foundation, will cost about $4,000. The cost in 1903 of a reinforced concrete stack 150 ft. high, 6 ft. inside diameter, including foundation, was $3,800. The contract price in 1907 of a reinforced concrete stack 166 ft. high, 8 ft. inside diameter, was $4,100.

The cost of a 118,000-gallon concrete-steel stand pipe, with enclosing tower, at Hull, Mass. was about $12,000. (See Eng. News, Vol. 52, p. 596.)

Mr. H. G. Tyrrell, in R. R. Gazette, Dec. 30, 1904, shows that the cost of the parts of single track steel trestle for E50 loading, towers 30 ft. between bents, at 3½ cts. per lb. for girders, 4 cts. per lb. for bents and bracing, and $10 per cu. yd. for concrete, are as follows:

Length intermediate Span	Cost of Steel Trestle 120 Ft. High per lin. ft. Spans	Bents	Traction Bracing	Piers	Total
30	$15.15	$45.2	$15.2	$12.0	$87.55
60	21.77	39.2	13.0	8.0	82.57
100	39.09	32.0	10.8	5.5	87.39

In Proceedings, Am. Ry. Eng. M. of W. Asso. Vol. 2, p. 139 it is stated that the cost of ballasted trestle on the A. T. & S. Fe Ry., 2 examples, averaged $12.66 per lin. ft. divided as follows: treated piles, $5.02; lumber, $5.01; bolts, $.21; cross ties, $.24; ballast, $.285; labor (all kinds), $1.89; creosote, $.005.

Mr. J. C. Bland, in 1891, found the estimated contract price of single track timber trestle was, in round numbers, nearly equal in dollars per lineal foot to one-half of the height of trestle in feet out to out of cap and sill. In this the floor deck alone was $4.22 per ft. of track and piles $3 each; both increased by 20% for the contractor's profit.

Timber in place was estimated at $52 to $55 per M. B. M., contract price.

A number of examples of the cost of railroads are given in R. R. Gazette, Sept. 7, and Oct. 26, 1906, as follows: 1st case. No tunnels, few bridges, along river, considerable cut and fill, single track, $26,300 per mile. 2d case. Along river, heavy cuts, some bridges, single track, $37,014 per mile. 3d case. Cuts and fills, bridges, tunnels, single track, $60,628 per mile. 4th case. Heavy crossings, double track, $76,336 per mile. 5th case. Heavy crossings, double track, $105,186 per mile. 6th case. Detour around large city, double track, $50,000 per mile. In the foregoing the cost includes preliminary surveys, clearing right of way, roadbed, ties, rails, ballast, side tracks, but does not include real estate, stations equipment, or signals.

Following is a list, alphabetically arranged, giving prices of various materials and work, to be used as a guide in estimating the cost of structures. These are taken largely from current price lists and contract prices, as published in engineering journals, and in general indicate prices prevailing in 1904 to 1907.

ASPHALTUM: Ventura and other California asphalts, $20 to $23 per ton at New York; Trinidad refined, $22 to $25 per ton; Venezuela asphalt, $25 to $60 per ton; Bermuda asphalt, $25 to $35.

BRICK: At yards, per thousand, common soft, $5 to $7; hard $7 to $9; vitrified (hard burned), paving, common, $8 to $12, special, $15 to $20; select red, not pressed, $8 to $10, pressed, $14 to 18; Roman, $30; fire bricks, $14. Freight on bricks is about $2 per thousand for 50 mile run.

BRONZE: Phosphor, in place, abt. 40 cts. lb.

CAST IRON: Pig Iron, $19 to $22 per long ton.

Cast iron counterweights, 1½ to 2 cts. per lb. delivered.

Cast iron pipe, $33 to $38 per ton delivered; laid, $38 to $45 per ton.

Standard and plain castings, 2½ to 3½ cts. per lb. in place. *Special castings,* large orders, 3 to 5 cts. per lb. *in place; small orders* 5 to 10 cts.

CEMENT: Portland $1.50 to $2.00 per bbl., 400 lbs.

Rosendale, 80 cts. to $1 per bbl., 300 lbs.

Large users of Portland cement pay less than $1.50 per bbl. for domestic brands. On small orders freight and handling increase the cost.

CEMENT FINISH: Portland, mortar ½ in. thick 50 to 80 cts. per sq. yd.

CLAY: Fireclay, dry powder $1.50 ton delivered on cars; calcined fireclay, $3 to $4 ton.

For puddle $1.50 cu. yd. delivered.

CONCRETE: Natural cement, $3 to $5 cu. yd. in place.

Portland cement, in large mass, easily deposited, $4 to $7 cu. yd. Walls requiring difficult forms, $6 to $8 cu. yd. Tunnels, etc. $10 to $12 cu. yd.

The cost of 1:3:6 Portland cement concrete may be analyzed as follows:

1 cu. yd. broken stone	$2.00
½ cu. yd. sand	.50
1 bbl. cement	2.00
Mixing and depositing	.50
Total	$5.00

This is with the use of a mechanical mixer. Hand mixing would probably cost from 70 cts. to $1.25 per cu. yd.

For detailed information on the cost of concrete structures on the N. C. & St. L. R. R. see paper by H. M. Jones, published in part in the R. R. Gazette, Oct. 21, 1904. The cost to the R. R. Co. per cubic yard for culverts and walls run about $6 to $7. Two examples are a little over $9.

Reinforced concrete, including steel, usually costs from $10 to $20 per cu. yd. Concrete should be estimated at $5 to $10 per cu. yd. in place, steel at about 2.5 cts per lb. in place (plain structural steel), forms 5 to 10 cts. per sq. ft. The unit cost of concrete will depend upon the difficulty of handling and placing.

COPPER: 14 to 15 cts. lb.

CURB: Cement and sand, 1:3, 25 to 50 cts. per lineal *foot, about ½ ct.* per sq. in. of section per lineal foot, in *place.*

Sandstone and limestone, 50 cts. to $1 per lineal foot, in place.

Bluestone, $1 to $1.50 per lineal ft. in place.

Granite, $1 to $2,50, lin. ft. in place.

Curved curbs in stone 20% to 100% extra.

Resetting curb 10 to 50 cts. per ft.

DREDGING: Soft material 12 to 30 cts. per cu. yd.; gravel and hard material 30 cts. to $1 cu. yd. In Eng. News, Sept. 20, 1906, a report is given of some dredging done by U. S. Govt. Engineers with hydraulic dredges (New York Harbor) which cost only 5.274 cts. per cu. yd.

EXCAVATING: In earth, large masses, above water, 25 to 50 cts. cu. yd., below water, for piers, $1 to $5 cu. yd.; in trench, earth, 50 cts. to $1 cu. yd.; loose rock, $1 to $2 cu. yd.; hard rock, $1 to $3 cu. yd.

Steam shovel work costs about 12 to 20 cts per cu. yd. In Eng. Record, Vol. 54, p. 732, some data are given from a paper by Mr. John C. Sessor on steam shovel work on the C. B. & Q. Ry. On one job of 251,711 cu. yds. 1,104 cu. yds. were moved per 10-hr. shift. The cost was as follows: equipment, 1 cent; steam shovel service, 8.9 cents; temporary trestle, 3.6 cents; track and track work, 5 cents: supervision and engineering 0.2 cents; total, 18.7 cents, all per cubic yard. On another job of 188,240 cu. yds. 946 cu. yds. were moved per 10-hr. shift. The cost was as follows: equipment, 1½ cents; steam shovel service, 9.6 cents; temporary trestle, 3.1 cents; track and track work, 4.2 cents; supervision and engineering, 0.3 cents; total, 18.7 cents, all per cubic yard.

The I. C. R. R. estimates excavating in earth, in jobs below 50,000 cu. yds., to cost 25 cts. per cu. yd., and in larger jobs, 20 cts. per cu. yd. adding in both cases one cent per cu. yd. per 100-ft. haul.

A committee report of the Roadmaster's and Maintainance-of-Way Asso., published in the R. R. Gazette of Oct. 31, 1904, and in Eng. News of Oct. 27, 1904, gives the following *as the cost of ditching* cuts and widening *embankments.*

By wheelbarrows: 12.2 cts. per cu. yd. plus 31 cts. per cu. yd. per 1000 ft. haul for common loam or 7.3 cts. extra in bad, wet material.

By push cars: 19.1 cts. per cu. yd., where material is unloaded by shovel, or 15.9 cts. where unloaded by dumping box, or similar arrangement, plus 33.4 cts. per cu. yd. per 5,000 ft. haul.

By machine ditcher: 22 to 30 cts. per cu. yd., the latter figure being for a 15-mile haul in loam. In wet or bad material add about 4.5 cts. per cu. yd.

The same report places the cost of team work with scrapers at 14 to 25 cts. per cu. yd., and of ditching by casting, in fair digging, where one cast will place the material in suitable final location, at 10 cts. per cu. yd. Much valuable information is given in this report.

FENCE: Board, 50 cts. to $1.50 per ft.

FILLING: Earth, material at hand, 20 cts. to 50 cts. cu. yd.

FLAG STONE: In place $1 to $3 sq. yd.

FORMS: Allow 5 to 10 cts. per sq. ft. for concrete forms, depending on whether lumber is dressed or not and on number of times it can be used.

FRENCH DRAIN: 50 cts. to $1 lin. ft.

FUEL: Hoisting engines, etc. Allow 1/3 ton of coal per 10 horse power per 10 hr. shift. (Gillette.)

GRAVEL: In bank, 15 to 20 cts. cu. yd., f. o. b. cars 35 to 40 cts. cu. yd., freight for 50-mile run, about 75 cts. cu. yd., hauling, 25 to 50 cts. cu. yd. Usual price delivered about $1 cu. yd.

LIME: Common, bbl. (250 lbs.) 80 cts., finishing $1; per ton at works, $3.75, delivered, $6.

LEAD: Pig, about 4.6 cts. lb.; lead pipe about 5 cts. lb.

MASONRY: Rubble, dry, $2 to $5 cu. yd., in mortar, $3 to $8 cu. yd. Coursed rubble, large stones, $5 to $8.

Brick, common, $6 to $10 cu. yd., good, $10 to $15 cu. yd. Laying brick, $2 to $3 cu. yd. Cost of lime mortar per cu. yd. of brick work about 60 cts., of cement mortar $1 to $2.

On the basis of $7.25 per M. for red brick, $2.50 per bbl. for cement, $1.25 per bbl. for lime, $1.25 per cu. yd. of sand, assuming a mason at 65 cts. per hr. with help at [illegible]½ cts. per hr. to lay 1,200 bricks in 8 hrs., a brick wall [illegible] ins. thick will cost about 40 cts. per superficial foot. With pressed brick face the cost will be about 50 cts. per superficial foot.

Bridge pier, sandstone or limestone, $8 to $12 cu. yd.

Ashlar, sandstone or limestone, $12 to $20 cu. yd., granite, $20 to $30 cu. yd.

Dressed bluestone, for steps etc., $1 to $2 cu. ft.

MINERAL WOOL: Slag, ordinary, sh. ton, $19; selected, $25; rock, ordinary, $32; selected, $40.

PAINT: Prepared, $1 to $1.50 gallon.

PAVING: Asphalt—In 44 cities in North America the cost of asphalt paving including 4 to 6 ins. of concrete, 1 to 1½ ins. of binder, and 1½ to 2 ins. of surface, in 1900 varied between $1.43 and $3.25 per sq. yd. (See Eng. Record, Vol. 43, No. 8.) It is estimated that the cost of guarantee for the first five years is 3 cts. per yard and for the second five years is 15 cts. per yard. The congresional appropriation bill allowed $1.80 per sq. yd. to be paid for asphalt pavements in Washington, D. C. (Eng. News, Aug. 13, 1903.)

Asphalt block, $2 to $2.50 sq. yd.

The division of the cost of asphalt pavement is about as follows: 2½" of surface, 67 cts. sq. yd.; 2" of binder 13 cts. sq. yd.; 6" of Portland cement concrete $1. Total $1.80 sq. yd.

Brick, work only, 15 to 20 cts. sq. yd.

Brick, 4" of brick on 3" of sand, 65 to 85 cts. sq. yd.; 4" of brick on 6" of natural cement concrete and 1½" cushion of sand, $1.20 to $1.60 sq. yd.; sidewalks, 2" of brick on sand 50 to 80 cts. sq. yd.

Cobble stone, 80 cts. sq. yd.

Concrete sidewalks, finished with mortar of sand and cement, granite screenings and cement, etc. 10 to 25 cts. sq. ft. Mortar finish alone 5 to 15 cts. sq. ft.

A common contract price for concrete sidewalks, small jobs, is 15 to 20 cts. per sq. ft. Large paving work can be done at an actual cost of about 10 cts. per sq. ft.

Macadam, stone free at quarry, 8" depth, 40 to 50 cts. sq. yd.; including cost of stone, 8" depth, 60 to 90 cts. sq. yd., 12" depth, 90 cts. to $1.30 sq. yd.

Stone blocks on broken stone base, $1.50 to $2 sq. yd.

Stone blocks on concrete base $2 to $3.50 sq. yd.

Wooden blocks—4 in. creosoted yellow pine blocks on one inch of sand over 6 in. of natural cement concrete $2.25 to $2.35 per sq. yd. Cost of 4" creosoted yellow pine blocks f. o. b. cars about $1.70 per sq. yd.

The following analyses of the cost of brick and stone block paving are taken from Engineering News, July 24, 1902.

"The following is a summary of the cost of paving with brick laid on edge, wages being 25 cts. per hour for pavers and 15 cts. for laborers:

	Cost per sq. yd.
57 "pavers" at $10 per M.	$0.57
Hauling 1½ miles over earth roads	.06
Laying pavers, including labor of grouting	.08
0.18 cu. ft. = 1-150 cu. yd. of grout*	.05
1-36 cu. yd. sand cushion at $1.08 a cu. yd.	.03
Plank to protect concrete	.01
Total net cost	$0.80
Add about 19% for profit	.15
Contract price	$0.95

* 1 Portland to 2 sand.

"To this, of course, must be added the cost of grading and cost of concrete foundation.

On block paving "we have for the total labor cost:

	Per sq. yd.
Loading and unloading inclusive of lost team time	$0.10
Hauling 1 mile	.05
Distributing blocks	.03
Laying	.06

Filling joints	.06
Foreman at 40 cts. per hr., 30 sq. yds.	.013
2 water and errand boys	.007
Total labor	$0.30

"Cost of Medina Block Pavement.

	Per sq. yd.
1/3 cu. yd. street excavation	$0.15
6-in. concrete foundation	.50
1-18 cu. yd. sand cushion in place at $1.08	.06
Medina block (6 in.) f. o. b. Albion, N. Y.	1.15
Freight to Rochester	.07
Unloading, hauling and laying	.30
1.5 gallons tar at 10 cts. a gallon	.15
1-50 cu. yd. sand for joints	.02
Total	$2.40
Add for contractor's profit	.25
Total cost	$2.65

Cost of street paving in 30 cities in Wisconsin per sq. yd. (See Municipal Journal and Engineer, Nov., 1905) asphalt $1.80 to $2.19; bricks, $1 to $2.19; macadam $.25 to $1.30; wood block $.60 to $1.97.

All-concrete roadway paving has been found in several cities to cost 14 to 18 cts. per sq. ft. At Jackson, Mich., some street paving having 3 ins. of gravel; 6 ins. of 1:8 cement and gravel; 4 ins. of 1:3 cement and 1/2-in. crushed granite, mixed quite wet, cost 18 cts. per sq. ft. (See Concrete Engineering, Dec., 1907, p. 205.)

PILES: Driven and cut, ordinary lengths and sizes, spruce 20 to 40 cts. ft.; white oak, 25 to 60 cts. ft. Spruce 30 to 40 ft. long, driven and cut, $6 to $10 each. Shorter piles for trestle bents $3 to $5 in place.

PILING: (Nov., 1907) Spruce, ordinary cargoes, 6 to 7 cts. ft. Oak, 14-in. butt, 40 to 50 ft., 19 cts. ft.; 50 to 55 ft., *22 cts. ft.; 55 to 60* ft., 23 cts. ft.; 60 ft. and up, 25 *cts. ft.*

Pine, 60 to 65 ft., $8.50 each; 70 to 75 ft. $10.50 each; 80 ft. and up, $16 each.

Concrete piles in place, about $1 per lineal foot.

PIPE: Vitrified pipe, 8" dia., 15 cts., hauling ½ ct. ft., laying, 1½ cts. ft., cement, ½ ct. ft. = 17.5 cts. ft. in trench already dug. For 12" pipe the cost is about 35 cts. per ft. total. (See Eng. Record, March 10, 1906, p. 350.)

RAILING: Gaspipe, 2-rail, 50 to 75 cts. ft., 3-rail, 75 cts. to $1.25 ft.

A substantial bridge railing costs about $1.50 to $2.50 per lineal ft. Cast iron newel-posts about $10 each.

ROOFING: Four layers of felt paper covered with pitch and gravel or pitch and slag 4 to 6 cts. sq. ft.

Slate, 7 to 13 cts. per sq. ft. Slag, 4 cts. Tin, 8 to 10 cts. Shingle, 7 to 10 cts. per sq. ft.

Tile, Spanish $9 to $12 per square of 100 sq. ft.

SAND BLASTING: (Cleaning) Large contracts 1 to 3 cts. sq. ft. (See Eng. News, Vol. 47, Page 324.)

SAND: Building, 20 to 25 cts. cu. yd. in bank, f. o. b. cars, 40 to 50 cts. cu. yd. freight for 50-mile run, about 75 cts. cu. yd., hauling, 25 to 50 cts. cu. yd. Usual price delivered about $1.10 cu. yd.

SEEDING: In grass $25 to $75 acre.

SEWER PIPE: Laying and cementing in trench already dug, small sizes, 15 to 25 cts. per ft., large sizes 50 cts. to $1 per ft.

Cost of pipe per lineal ft., 5", 5 to 7 cts., 10", 15 to 20 cts. 15", 25 to 40 cts., 21", 50 to 75 cts., 30", $1 to $1.70; 48", $2 to $3.

SHRUBS: 50 cts. to $2 each.

SODDING: In country towns, 7 to 10 cts. sq. yd., in cities, 25 to 50 cts. sq. yd.

STEEL: Structural, material only, 1¾ to 2¼ cts. per lb. Erected and painted, 3 to 5 cts. per lb.

Castings, in place, 5 to 10 cts. lb.

Rails, new, $28 ton, f. o. b. cars; old, short pieces, $14 *to $14.50.*

Scrap, structural, $14 long ton.

STONE: Wholesale rates, delivered at N. Y.; price per cu. ft.

Nova Scotia, in rough, 90 cts.; Ohio freestone, in rough, 85 to 90 cts.; Minnesota freestone, in rough, 90 cts.; Longmeadow freestone, 85 cts.; Brownstone, Portland, ct., 60 cts.; Brownstone, Belleville, N. J., 75 cts. to $1; Scotch redstone, $1.05; Lake Superior redstone, $1.10; granite, rough, 40 to 50 cts.; limestone, buff and blue, 80 cts.; portage, $1; Caen, $1.25 to $1.75; white building marble, $1.25 to 1.75; Wyoming bluestone, 90 cts.; Euclid bluestone, 90 cts.; crushed stone, $1.40 per net ton. f. o. b. cars N. Y. C. (May, 1904.)

TARRED PAPER: 1 ply (roll 300 sq. ft.), ton, $32.50 to $35.50; 2 ply, roll 108 sq. ft., 55 to 60c roll; 3 ply, roll 108 sq. ft., 78 to 80c. Slater's Felt (roll 500 sq. ft.) 75c. R. R. M. Stone Surfaced Roofing (roll 110 sq. ft.) $2.75.

TAR: Regular bbl., $2.25, oil bbl., $5.75; Coal tar, gallon, 8 cts.

TIES, RAILROAD: Untreated, cedar and spruce, 40 to 60 cts. each; oak and yellow pine, 60 to 80 cts. each. See Trans. Am. Ry. Engineering and M. of W. Asso. Vol. 2 for cost of ties to 13 of the principal American railroads. It is there shown to vary between 35½ and 81½ cents each.

TOOLING: Bush hammering concrete surfaces 2 to 5 cts. per sq. ft.

TRANSPORTING: The cost of picking up materials such as stone or sand and hauling them a moderate distance in wheelbarrows is about 20 to 25 cts. per cu. yd. With wagons or carts the cost is about 15 to 23 cts. per cu. yd.

TREATING WOOD: Cheaper processes 5 to 10 cts. per cu. ft. Creosoting 20 to 60 cts. per cu. ft.

Railroad ties 20 cts. each, up.

Creosoted yellow pine in place costs, including cost of *wood, $65 to $80* per 1,000 ft. B. M.

TREES: $1 to $5 each.

WOOD: (Prices per thousand feet of board measure) May, 1904.

Hemlock, rough, in lengths up to 20 ft. $17 to $19. Lengths 22 to 40 ft. $3.25 to $7 additional.

Pine, yellow (Long Leaf) building orders, 12 ins. and under, $20.50 to $22.50; 14-in. and up, $26 to $29; 1¼ and 1½-in. wide boards, $28 to $30; 2-in. wide plank, heart face, $30 to $31.50.

Yellow pine of heavy construction, in cargo lots, delivered New York City, $22 to $25.

Spruce: random cargoes, 2-in. cargoes, $18 to $21; 6 to 9-ins., cargoes $19.50 to $21.50; 10 and 12-in. cargoes, $21 to $23.

The framing and placing of wood in a structure costs $5 to $15 per thousand feet of board measure.

White oak timber in wharf construction costs $50 to $60 per thousand feet in place.

Bridge timber in place, per thousand feet, white oak, $40 to $50; yellow pine, $35 to $45; hemlock, $22 to $30.

More than half the cost of wood is generally due to freight on account of the long distances between the centers of greatest supply and greatest consumption.

Specifications for Natural and Portland Cement.*

Recommended Standard Specifications.

(Standard Specifications for Cement adopted by a Joint Committee embracing representatives from the American Society of Civil Engineers, American Society for Testing Materials, American Institute of Architects, Engineer Department of United States Army, Association of Portland Cement Manufacturers and American Railway Engineering and Maintenance of Way Association.)

General Observations.

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

3. Specific gravity is useful in detecting adulteration or underburning. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

4. The sieves should be kept thoroughly dry.

5. Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which

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* The specifications and recommendations that follow are taken without change, except the omission of figures showing apparatus and reference to same, from *Manual of Recommended Practice*, Edition of 1907, published by American *Railway* Engineering and Maintenance of Way *Association.*

the tests are made, a very dry or humid atmosphere, and other irregularities vitally affect the rate of setting.

6. Each consumer should fix the minimum requirements for tensile strength to suit his own conditions. They shall, however, be within the limits stated.

7. The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

8. In making the pats the greatest care should be exercised to avoid initial strains due to molding or too rapid drying out during the first twenty-four hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

9. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement may, however, be held for twenty-eight days, and a retest made at the end of that period. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

STANDARD SPECIFICATIONS FOR CEMENT

GENERAL CONDITIONS.

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, cement shall be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to *permit* easy access for proper inspection and identification *of each shipment.*

5. *Every facility* shall be provided by the contractor and

a period of at least twelve days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain four bags, and each barrel of Natural cement shall contain three bags of the above net weight.

8. Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society January 21, 1903, and amended January 20, 1904, with all subsequent amendments thereto (See addendum to these specifications.)

10. The acceptance or rejection shall be based on the following requirements:

NATURAL CEMENT.

11. This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

12. The specific gravity of the cement, thoroughly dried at 100° C., shall not be less than 2.8.

13. It shall leave by weight a residue of not more than 10 per cent. on the No. 100, and 30 per cent. on the No. 200 sieve.

14. It shall develop initial set in not less than ten minutes, and hard set in not less than thirty minutes, nor more than three hours.

15. The minimum requirements for tensile strength for *briquettes one inch* square in cross-section shall be within

the following limits, and shall show no retrogression in strength within the periods specified:*

Age.	*Neat Cement.*	Strength
24 hours in moist air	....	50-100 lbs.
7 days (1 day in moist air, 6 days in water)	..	100-200 lbs.
28 days (1 day in moist air, 27 days in water)		200-300 lbs.
	One Part Cement, three Parts Standard Sand.	
7 days (1 day in moist air, 6 days in water)		25- 75 lbs.
28 days (1 day in moist air, 27 days in water)		75-100 lbs.

16. Pats of neat cement about three (3) in. in diameter, one-half (½) in. thick at center, tapering to a thin edge, shall be kept in moist air for a period of twenty- four hours.

(a) A pat is then kept in air of normal temperature.

(b) Another is kept in water maintained as near 70° F. as practicable.

17. These pats are observed at intervals for at least 28 days, and, to satisfactorily pass the tests, should remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

PORTLAND CEMENT.

18. This term shall be applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent. has been made subsequent to calcination.

19. The specific gravity of the cement, thoroughly dried at 100° C., shall be not less than 3.10.

20. It shall leave by weight a residue of not more than 8 per cent. on the No. 100, and not more than 25 per cent. on the No. 200 sieve.

21. It shall develop initial set in not less than thirty

* For example, the minimum requirement for the twenty-four hour neat cement tests should show some specified value within the limits of 50 and 100 pounds, and so on for each period stated.

minutes, but must develop hard set in not less than one hour. nor more than ten hours.

22. The minimum requirements for tensile strength for briquettes one-inch square in cross-section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	Strength.
24 hours in moist air		150-200 lbs.
7 days	(1 day in moist air, 6 days in water)	450-550 lbs.
28 days	(1 day in moist air, 27 days in water)	550-650 lbs.
	One Part Cement, Three Parts Standard Sand.	
7 days	(1 day in moist air, 6 days in water)	150-200 lbs.
28 days	(1 day in moist air, 27 days in water)	200-300 lbs.

23. Pats of neat cement about three (3) in. in diameter, one-half (½) in. thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(a) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(b) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least 28 days.

(c) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

24. These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

25. The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO^3), nor more than 4 per cent. of magnesia (MgO).

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* For example, the minimum requirement for the twenty-four hour neat cement tests should show some specified value within the limits of 150 and 200 pounds, and so on for each period stated.

ADDENDUM.

ABSTRACT OF METHODS RECOMMENDED BY THE SPECIAL COMMITTEE ON UNIFORM TESTS OF CEMENT OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

SAMPLING.

1. The sample shall be a fair average of the contents of the package; it is recommended that, where conditions permit, one barrel in every ten be sampled.

2. All samples should be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material; this is also a very effective method for mixing them together in order to obtain an average. For determining the characteristics of a shipment of cement, the individual samples may be mixed and the average tested; where time will permit, however, it is recommended that they be tested separately.

3. Cement in barrels should be sampled through a hole made in the center of one of the staves, midway between the heads, or in the head, by means of an auger or a sampling iron similar to that used by sugar inspectors. If in bags, it should be taken from surface to center.

CHEMICAL ANALYSIS.

4. As a method to be followed for the analysis of cement, that proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, of the New York Section of the Society for Chemical Industry, and published in the *Journal* of the Society for January 15, 1902, is recommended.

SPECIFIC GRAVITY.

5. The determination of specific gravity is most conveniently made with Le Chatelier's apparatus. This consists of a flask of 120 cu. cm. (7.32 cu. in.) capacity, the neck of which is about 20 cm. (7.87 in.) long; in the middle of this neck is a bulb, above and below which are two marks; the volume between these marks is 20 cu. cm. (1.22 cu. in.) The neck has a diameter of about 9 mm. (0.35 in.), and is graduated into tenths of cubic centimeters above the mark.

6. Benzine (62° Baume naptha), or kerosene free from water, should be used in making the determination.

7. The specific gravity can be determined in two ways:

(a) The flask is filled with either of these liquids to the lower mark and 64 gr. (2.25 oz.) of powder, previously dried at 100° C. (212° F.) and cooled to the temperature of the liquid, is gradually introduced through the funnel [the stem of which extends into the flask to the top of the bulb], until the upper mark is reached. The difference in weight between the cement remaining and the original quantity (64 gr.) is the weight which has displaced 20 cu. cm.

8. (b) The whole quantity of the powder is introduced, and the level of the liquid rises to some division of the graduated neck. This reading plus 20 cu. cm. is the volume displaced by 64 gr. of the powder.

9. The specific gravity is then obtained from the formula:

$$\text{Specific Gravity} = \frac{\text{Weight of Cement}}{\text{Displaced Volume}}$$

10. The flask, during the operation, is kept immersed in water in a jar, in order to avoid variations in the temperature of the liquid. The results should agree within 0.01.

11. A convenient method for cleaning the apparatus is as follows: The flask is inverted over a large vessel, preferably a glass jar, and shaken vertically until the liquid starts to flow freely; it is then held still in a vertical position until empty; the remaining traces of cement can be removed in a similar manner by pouring into the flask a small quantity of clean liquid and repeating the operation.

FINENESS.

12. The sieves should be circular, about 20 cm. (7.87 in.) in diameter, 6 cm. (2.36 in.) high, and provided with a pan 5 cm. (1.97 in.) deep, and a cover.

13. The wire cloth should be woven (not twilled) from brass wire having the following diameters:

No. 100, 0.0045 in.; No. 200, 0.0024 in.

14. This cloth should be mounted on the frames without

distortion; the mesh should be regular in spacing and be within the following limits:

No. 100, 96 to 100 meshes to the linear inch.

No. 200, 188 to 200 meshes to the linear inch.

15. Fifty grams (1.76 oz.) or 100 gr. (3.52 oz.) should be used for the test, and dried at a temperature of 100° C. (212°F.) prior to sieving.

16. The thoroughly dried and coarsely screened sample is weighed and placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward, at the same time striking the side gently with the palm of the other hand, at the rate of about 200 strokes per minute. The operation is continued until not more than one-tenth of 1 per cent, passes through after one minute of continuous sieving. The residue is weighed, then placed on the No. 100 sieve and the operation repeated. The work may be expedited by placing in the sieve a small quantity of large shot. The results should be reported to the nearest tenth of 1 per cent.

NORMAL CONSISTENCY.

17. This best can be determined by means of *Vicat Needle Apparatus,* which consists of a frame, bearing a movable rod with a cap at one end, and at the other a cylinder, 1 cm. (0.39 in.) in diameter, the cap, rod and cylinder weighing 300 gr. (10.58 oz.). The rod, which can be held in any desired position by a screw, carries an indicator, which moves over a scale (graduated to centimeters) attached to the frame. The paste is held by a conical hard-rubber ring, 7 cm. (2.76 in.) in diameter at the base, 4 cm. (1.57 in.) high, resting on a glass plate about 10 cm. (3.94 in.) square.

18. In making the determination, the same quantity of cement as will be subsequently used for each batch in making the briquettes [but not less than 500 gr. (17.16 oz.)] is kneaded into a paste, as described in paragraph 38, and *quickly formed into* a ball with the hands, completing the *operation by tossing* it six times from one hand to the

other, maintained 6 in. apart; the ball is then pressed into the rubber ring, through the larger opening, smoothed off and placed (on its large end) on a glass plate and the smaller end smoothed off with a trowel; the paste, confined in the ring, resting on the plate, is placed under the rod bearing the cylinder, which is brought in contact with the surface and quickly released.

19. The paste is of normal consistency when the cylinder penetrates to a point in the mass 10 mm. (0.39 in.) below the top of the ring. Great care must be taken to fill the ring exactly to the top.

20. The trial pastes are made with varying percentages of water until the correct consistency is obtained.

Note. The Committee of Standard Specifications for Cement inserts the following table for temporary use, to be replaced by one to be devised by the Committee of the American Society of Civil Engineers.

PERCENTAGE OF WATER FOR STANDARD MIXTURES.

Neat	1-1	1-2	1-3	1-4	1-5	Neat	1-1	1-2	1-3	1-4	1-5
18	12.0	10.0	9.0	8.4	8.0	33	17.0	13.3	11.5	10.4	9.6
19	12.3	10.2	9.2	8.5	8.1	34	17.3	13.6	11.7	10.5	9.7
20	12.7	10.4	9.3	8.7	8.2	35	17.7	13.8	11.8	10.7	9.9
21	13.0	10.7	9.5	8.8	8.3	36	18.0	14.0	12.0	10.8	10.0
22	13.3	10.9	9.7	8.9	8.4	37	18.3	14.2	12.2	10.9	10.1
23	13.7	11.1	9.8	9.1	8.5	38	18.7	14.4	12.3	11.1	10.2
24	14.0	11.3	10.0	9.2	8.6	39	19.0	14.7	12.5	11.2	10.3
25	14.3	11.6	10.2	9.3	8.8	40	19.3	14.9	12.7	11.3	10.4
26	14.7	11.8	10.3	9.5	8.9	41	19.7	15.1	12.8	11.5	10.5
27	15.0	12.0	10.5	9.6	9.0	42	20.0	15.3	13.0	11.6	10.6
28	15.3	12.2	10.7	9.7	9.1	43	20.3	15.6	13.2	11.7	10.7
29	15.7	12.5	10.8	9.9	9.2	44	20.7	15.8	13.3	11.9	10.8
30	16.0	12.7	11.0	10.0	9.3	45	21.0	16.0	13.5	12.0	11.0
31	16.3	12.9	11.2	10.1	9.4	46	21.3	16.1	13.7	12.1	11.1
32	16.7	13.1	11.3	10.3	9.5	-----	---	---	---	---	---

	1 to 1	1 to 2	1 to 3	1 to 4	1 to 5
Cement ------	500	333	250	200	167
Sand ---------	500	667	750	800	833

TIME OF SETTING.

21. For this purpose the Vicat Needle, which has already been described in paragraph 17, should be used.

22. *In making the test,* a paste of normal consistency *is molded and placed* under the rod, as described in para-

graph 18; this rod, bearing the cap at one end and the needle, 1 mm. (0.039 in.) in diameter, at the other, weighing 300 gr. (10.58 oz.). The needle is then carefully brought in contact with the surface of the paste and quickly released.

23. The setting is said to have commenced when the needle ceases to pass a point 5 mm. (0.20 in.) above the upper surface of the glass plate, and is said to have terminated the moment the needle does not sink visibly into the mass.

24. The test pieces should be stored in moist air during the test; this is accomplished by placing them on a rack over water contained in a pan and covered with a damp cloth, the cloth to be kept away from them by means of a wire screen; or they may be stored in a moist box or closet.

25. Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point reduces the area and tends to increase the penetration.

26. The determination of the time of setting is only approximate, being materially affected by the temperature of the mixing water, the temperature and humidity of the air during the test, the percentage of water used, and the amount of molding the paste receives.

STANDARD SAND.

27. For the present, the Committee recommends the natural sand from Ottawa, Ill., screened to pass a sieve having 20 meshes per linear inch and retained on a sieve having 30 meshes per linear inch; the wires to have diameters of 0.0165 and 0.0112 in., respectively, *i. e.*, half the width of the opening in each case. Sand having passed the No. 20 sieve shall be considered standard when not more than 1 per cent. passes a No. 30 sieve after one minute continuous sifting of a 500-gr. sample.

28. The Sandusky Portland Cement Company of Sandusky, Ohio, has agreed to undertake the preparation of this sand and to furnish it at a price only sufficient to cover the actual cost of preparation.

FORM OF BRIQUETTE.

29. While the form of the briquette recommended by a former Committee of the Society is not wholly satisfactory, this Committee is not prepared to suggest any change, other than rounding off the corners by curves of ⅛-in. radius.

MOLDS.

30. The molds should be made of brass, bronze or some equally non-corrodible material, having sufficient metal in the sides to prevent spreading during molding.

31. Gang molds, which permit molding a number of briquettes at one time, are preferred by many to single molds; since the greater quantity of mortar than can be mixed tends to produce a greater uniformity in the results.

32. The molds should be wiped with an oily cloth before using.

MIXING.

33. All proportion should be stated by weight; the quantity of water to be used should be stated as a percentage of the dry material.

34. The metric system is recommended because of the convenient relation of the gram and the cubic centimeter.

35. The temperature of the room and the mixing water should be as near 21° C. (70° F.) as it is practicable to maintain it.

36. The sand and cement should be thoroughly mixed dry. The mixing should be done on some non-absorbing surface, preferably plate glass. If the mixing must be done on an absorbing surface is should be thoroughly dampened prior to use.

37. The quantity of material to be mixed at one time depends on the number of test pieces to be made; about 1,000 gr. (35.28 oz.) makes a convenient quantity to mix, especially by hand methods.

38. The material is weighed and placed on the mixing table, and a crater formed in the center, into which the proper *percentage of* clean water is poured; the material *on the outer edge is* turned into the crater by the aid of a

trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for an additional $1\frac{1}{2}$ minutes, the process being similar to that used in kneading dough. A sandglass affords a convenient guide for the time of kneading. During the operation of mixing the hands should be protected by gloves, preferably of rubber.

MOLDING.

39. Having worked the paste or mortar to the proper consistency, it is at once placed in the molds by hand.

40. The molds should be filled at once, the material pressed in firmly with the fingers and smoothed off with a trowel without ramming; the material should be heaped up on the upper surface of the mold, and, in smoothing off, the trowel should be drawn over the mold in such a manner as to exert a moderate pressure on the excess material. The mold should be turned over and the operation repeated.

41. A check upon the uniformity of the mixing and molding is afforded by weighing the briquettes just prior to immersion, or upon removal from the moist closet. Briquettes which vary in weight more than 3 per cent. from the average should not be tested.

STORAGE OF THE TEST PIECES.

42. During the first 24 hours after molding, the test pieces should be kept in moist air to prevent them from drying out.

43. A moist closet or chamber is so easily devised that the use of the damp cloth should be abandoned if possible. Covering the test pieces with a damp cloth is objectionable, as commonly used, because the cloth may dry out unequally, and in consequence the test pieces are not all maintained under the same condition. Where a moist closet is not available, a cloth may be used and kept uniformly wet by immersing the ends in water. It should be kept from direct contact with the test pieces by means of a wire screen or some similar arrangement.

44. A moist closet consists of a soapstone or slate box

or a metal-lined wooden box—the metal lining being covered with felt and this felt kept wet. The bottom of the box is so constructed as to hold water, and the sides are provided with cleats for holding glass shelves on which to place the briquettes. Care should be taken to keep the air in the closet uniformly moist.

45. After 24 hours in moist air, the test pieces for longer periods of time should be immersed in water maintained as near 21° C. (70° F.) as practicable; they may be stored in tanks or pans, which should be of non-corrodible material.

TENSILE STRENGTH.

46. The tests may be made on any standard machine. A solid metal clip is recommended. This clip is to be used without cushioning at the points of contact with the test specimen. The bearing at each point of contact should be ¼-in. wide, and the distance between the centers of contact on the same clip should be 1¼ in.

47. Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the briquettes in the testing machine, as cross-strains, produced by improper centering, tend to lower the breaking strength. The load should not be applied too suddenly, as it may produce vibration, the shock from which often breaks the briquette before the ultimate strength is reached. Care must be taken that the clips and the sides of the briquette be clean and free from grains of sand or dirt which would prevent a good bearing. The load should be applied at the rate of 600 lbs. per minute. The average of the briquettes of each sample tested should be taken as the test excluding any results which are manifestly faulty.

CONSTANCY OF VOLUME.

48. Tests for constancy of volume are divided into two classes: (1) Normal tests, or those made in either air or water maintained at about 21° C. (70° F.), and (2) accelerated tests, or those made in air, steam or water at a *temperature of* 45° C. (115° F.) and upward. The test *pieces should* be allowed to remain 24 hours in moist air

before immersion in water or steam, or preservation in air.

49. For these tests, pats about 7½ cm. (2.95 in.) in diameter, 1¼ cm. (0.49 in.) thick at the center, and tapering to a thin edge, should be made, upon a clean glass plate [about 10 cm. (3.94 in.) square], from cement paste of normal consistency.

50. A pat is immersed in water maintained as near 21° C. (70° F.) as possible for 28 days, and observed at intervals. A similar pat is maintained in air at ordinary temperature and observed at intervals.

51. A pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel.

52. To pass these tests satisfactorily, the pats should remain firm and hard, and show no signs of cracking, distortion or disintegration.

53. Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

The Design of Concrete-Steel Beams and Slabs.

The formulas for proportioning concrete-steel beams are legion, and they are far more varied than those for the design of columns with which engineering literature was flooded a decade or two ago. Nice formulas for the design of columns were elaborated in the days before tests were so common that were satisfactory in every respect, except that they did not agree with the results of tests that were subsequently made. Investigators later found that the behavior of compression members of practical proportions agreed very closely with a simple little straight line formula. Even at the present time the last of these complex formulas has not been weeded out of specifications.

Another feature of construction that has yielded to the simplifying influence of development, or the developing influence of simplification, relates to the ultimate and the working stress of steel. The two grades of structural steel, with separate unit stresses, though they are still recognized in some specifications, are gradually merging into one grade for all pieces subject to the ordinary processes of the shop, excepting forging; and the manufacturer no longer needs to exhibit his ability to bring forth either grade from the same melt, bloom, billet or finished piece.

Concrete-steel designing is young, and probably on account of its rapid growth the infant is afflicted with both of the above-named ailments. The formulas brought out rival those for retaining walls in complexity, and the units recommended are about as numerous as the formulas.

The foregoing may seem an inconsistent preface to a paper suggesting both formulas and unit stresses, but the writer's purpose is to show the busy engineer, who does not want to delve into abstruse mathematics, that formulas for the design of concrete-steel beams or slabs can be *made as simple* as those for steel beams. In fact, the *formulas are* even more simple than formulas for steel

in bending, if certain rules of proportioning be adopted. Further, the agreement between the formulas and the actual strength of the construction is about as close as in steel construction, as many tests closely approximating the proportions here recommended have demonstrated.

In the matter of variation from an established unit stress we commonly allow steel to vary about 8 per cent either way from a desired average tensile strength. We should be satisfied if concrete-steel tests show as close agreement with their calculated strength, especially as the variation, barring the effects of careless work, will probably be on the safe side in nearly every case. The last-named condition obtains because of the real but uncertain element of strength imparted to the combination of concrete and steel by the strength possessed by the steel after it has passed its elastic limit, the nominal limit of ultimate strength.

There are many kinds of concrete, just as there are many grades of oak. Some oak has knots and wind shakes and is unfit for use in a structure, but specifications for oak beams do not recognize these grades in allowing unit stresses; rather the aim is to exclude those grades unfit for use by proper selection and inspection of the materials. So in concrete-steel construction there should be a standard, and this should be a concrete suitable for the purpose and made strictly according to the requirements, as near as these ends can be accomplished commercially.

It is almost universally agreed that concrete composed of 1 part Portland cement, 2 parts sand and 4 parts small hard broken stone or gravel, or of proportions closely approximating these, is the most suitable mixture for the concrete. For cinder concrete the same proportions, using cinders in place of broken stone, may be called standard. It is also common knowledge that this stone concrete, when made of good materials, will sustain an ultimate *crushing load of* about 2,000 lbs. per in., and the cinder *concrete will take* about 750 lbs. per sq. in. It is pre-

sumed in all cases that the materials and workmanship are good, just as we presume that wood used in structures is good and sound.

It is further almost generally agreed that the ultimate strength of a concrete-steel beam is reached when the steel is strained to its elastic limit and the concrete has reached its ultimate strength.

The use of steel having high elastic limit and dependence upon high safe units in consequence is indefensible. The excessive stretch in the steel will crack the concrete. The writer made a number of tests on the floors of a large concrete-steel building, and in one case, in a bay containing 13 beams, 91 cracks were counted in the beams when the "safe" load was placed upon the floor. One beam had 16 cracks. Many of these appeared long before the supposed safe load was placed. The deflection was excessive. This building was designed with steel of high elastic limit, and dependence was placed upon the high tensile value of the steel to sustain the loads.

The theoretic elongation of the concrete, even at moderate tensile values in the steel, corresponds to excessive tensile values in the concrete; and the fact that concrete in which steel is embedded has been stretched out in tests without cracking to elongations that would rupture plain concrete is evidence that the concrete in setting has shrunk, thus putting the steel under an initial compression, which must be overcome before any stretch occurs in the concrete. This shrinking of concrete in setting is one of its most useful properties, viewed as a medium in concrete-steel construction. Besides giving the embedded steel an initial compression and thus helping its tensile value, it also grips the steel, and thus takes firm hold upon it, greatly aiding the adhesion of the concrete to the steel. It is to be noted that this gripping of the steel, and not mere adhesion due to contact, is the thing that makes *the union between* the steel and the concrete effective. *This is the reason* why round and square bars hold better

in the concrete than flats. It is also a reason for bedding the steel deep enough from the surface of the concrete to make the gripping effective. It further overcomes the almost infinitesimal reduction in diameter in the embedded steel when under stress below the elastic limit, so that the adhesion or skin friction is not lost when the steel elongates slightly.

The fact that concrete shrinks in setting further points out the error in depending for compression upon steel embedded in concrete, unless the concrete is merely used for its protecting value. The initial compression in the steel adds an uncertain amount to any calculated compression in the same. Steel reinforcement in the intrados of segmental concrete floor arches is not rational design, as the steel, to be useful, must be in compression. Steel embedded in concrete columns as hoops or spirals is rational, as loading the column tends to increase its diameter and to put tensile stress in the steel. Also light longitudinal rods, wired to these spirals to take flexure in the columns, is excellent to reinforce the column against eccentric or lateral forces.

Another phase of this shrinking is that very long units in concrete-steel construction should not be placed at one time, unless expansion joints be provided. It is the shrinking of concrete that causes vertical cracks in plain concrete walls to appear at intervals, and makes it expedient to leave expansion joints in such walls 30 ft. apart or so. Steel embedded in concrete walls or other construction has the effect of distributing the shrinkage stresses and lessening the shrinkage cracks, and walls thus reinforced can be made much longer without liability to these cracks than plain walls. However, if a long reinforced wall is brought up uniformly from the ground up, shrinkage cracks will probably appear. If the wall be built from one end to the other, allowing setting and shrinkage of *part of the wall before* the remainder has been placed, a *very long wall can* be made without danger of these

cracks. By the same token long concrete-steel buildings should be placed in alternate units, if much is to finished at once, leaving the intermediate units to be placed after the setting of the first; or some other provision should be made to allow for the shrinkage of the concrete.

It is not the intention here to go into the subject of the making of concrete, more than to say that it is absolutely essential in this class of work that the concrete be thoroughly mixed, and it should be very wet. Dry or mealy concrete is totally unfit for concrete-steel work. It will neither adhere to the steel nor protect it.

Tests show that a rod embedded in wet concrete will resist pulling out, when the concrete has set and hardened, with a force of about 500 lbs. per sq. in. of the surface of rod embedded. At 10,000 lbs. per sq. in. on the steel and 50 lbs. per sq. in. adhesion to the concrete a round or square rod should be embedded 50 diameters in concrete. This is very often ignored in concrete-steel beams designed for buildings and prepared for test. The curve of maximum moments of a beam taking uniform load or a single rolling load is a parabola; hence the curve of stress in the reinforcing rods, assuming them to be parallel to the bottom of the beam, is a parabola. The line representing the adhesive value of a rod embedded in concrete is a straight line from the end of rod to the point of maximum stress; hence for the adhesive value to be at all sections greater than the actual stress the straight line should be outside of the parabola; that is, it should be tangent to it at the support. This would make the ordinate of the straight line just twice that of the parabola, or the adhesive value should be double the maximum stress. Therefore, it is essential in consistent design to have the beam no less than 100 times the diameter of rod from end to center of span. In other words, the rod should be no more than 1-200 of the span length.

Tensile value of the concrete should not be allowed *under any circumstance,* as one shrinkage crack may destroy *entirely this* tensile value for the whole beam.

As intimated, high tensile strains in the steel though they may be warranted from the standpoint of high elastic limit in the material, presume too far upon the extensibility of the concrete. The elastic limit of ordinary structural steel, or 40,000 lbs. per sq. in., is the most suitable value to represent the ultimate useful strength of the steel. With 2,000 lbs. per sq. in., as the ultimate strength of the concrete in compression, and with tension eliminated there remains the variation of the stress in the concrete from the neutral axis up and the position of the neutral axis to determine the value of the beam in flexure.

Tests have shown that the neutral axis of a concrete-steel beam remains close to the middle of the depth of the beam. A little calculation will show that a variation of one-eighth of the depth of beam one way or the other, in the standard construction here to be proposed, makes a difference of about 6 per cent in the calculated resisting moment, if the stress in the steel remains the same, or about the same difference if the extreme fiber stress in the concrete remains the same. Using the units heretofore given and 2,000,000 as the modulus of elasticity of concrete, and substituting in one of the elaborate formulas, which allows for parabolic variation of the stress in concrete from the neutral axis both ways and for tension in the concrete, the neutral axis is found to be within 4½ per cent of the middle of the depth of the beam. So that the neutral axis is shown, both by theory and test, to be close to the middle of the depth of the beam, and its shifting a comparatively large fraction of the depth makes but small variation in the calculated strength of the beam.

The assumption that the variation in stress in the concrete from the neutral axis to the extreme fiber is according to a curve deviating somewhat from a straight line is another element that complicates the formulas. Here, again, fine theory arrives at conclusions that are incompatible with the known variations in ultimate compressive strength of different tests made of the same materials

under apparently the same conditions. To assume that the intensity of stress in the concrete varies directly as the distance from the neutral axis is sound engineering and is accepted by many investigators.

In order to simplify further the calculations, it is recommended that the center of the steel be one-eighth of the depth from the bottom of the beam.

Referring to Fig. 1, it is seen that the total compression in the concrete is

$$2{,}000\ \frac{BD}{4} = 500\ BD$$

This must equal the tension in the steel. Now, if the area of the steel be A, we have $40{,}000\ A = 500\ BD$, or $A =$ 1.25 per cent of the rectangle BD.

The center of gravity of the stress in the concrete is

$$\frac{2}{3} \times \frac{D}{2} \text{ or } \frac{1}{3} D$$

above the neutral axis and the effective depth of beam is

$$\frac{1}{3} D + \frac{3}{8} D = \frac{17}{24} D.$$

The ultimate resisting moment is or

$$\frac{17}{24} D \times 500\ BD \text{ or } M = 354\ BD^2.$$

Assuming all dimensions in inches, this moment is in inch-pounds. Or, making $B = 12$ and dividing by 12 to reduce to foot-pounds, we have ultimate bending moment in foot-pounds per foot in width of slab or beam $= 354\ D^2$.

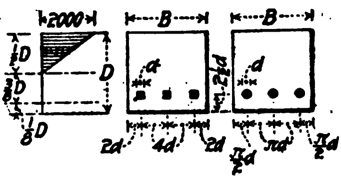


Fig. 1.

Now as to the shear. It is safe to say that any stone concrete that will not stand 50 lbs. per sq. in. as a safe load in shear should not enter into the construction of concrete-steel beams and slabs; also that concrete should not be stressed much above this amount in shear. The horizontal shear in the beam in a section just above the rods is equal at any point to the increment of stress in the rods. Hence, as we have used the same unit for shear and adhesion, there should be the same area across the beam as that of the surface of the rods. Round rods should then be spaced not less than 3.1416 times their diameter apart and square rods not less than four times their diameter apart; also the distance to the side of beam at last bar should not be less than one-half of these respective distances. With rods spaced just these amounts, it will be found that the distance from center of rod to bottom of beam will be 2½ times the diameter of rod, which is an ample depth to insure gripping of the steel by the concrete. Of course, rods can be given wider spacing than that shown in the figure. For example, square rods spaced five times their diameter apart will be two diameters from the bottom of beam.

This limit in the spacing of rods is very often overlooked in the designing of beams for buildings and for test. The beams are made too narrow and unscientific expedients are resorted to to overcome the defect. This has even more force in the case of dependence upon high tension steel or mechanical bond, as the greater increment of stress that must be assumed demands greater area of concrete to take the horizontal shear.

Again, in the matter of vertical shear at the end of span. Assuming a safe bending moment as one-fourth of the 354 D^2 already found, and equating to the expression for the bending moment in a uniformly loaded beam at W lbs. total load, we have

$$\frac{WL}{8}=88D^2 \text{ (} L=\text{span in feet)}$$

But the allowed end shear at 50 lbs. per sq. in. on a rectangle 12 ins. wide and D ins. deep, remembering that the maximum intensity of shear in a rectangular beam is three-halves of the average, is

$$\frac{W}{2}=\frac{2}{3}\times 50\times 12\ D,$$

or total allowed load on beam $= W = 800\ D$.

Substituting this value of W in the equation above, we have

$$D=\frac{100}{88}L.$$

But, as D is in inches and L is in feet, the actual ratio is very close to 10. Hence, when the depth is more than one-tenth of the span, the full load on a beam begins to overtax the shearing strength of the concrete before the steel reinforcement has its proper stress. Here again some investigators have erred by making short, deep test beams; and, finding that they fail in shear, concluding that concrete is inherently unreliable in shear; whereas the real fault is in the design of the beam.

In all concrete-steel designing more or less reliance must necessarily be placed upon the shearing strength of the concrete. It only remains to proportion the beam or slab in such way as to place the concrete where it will take the shear. The best mechanical bond can do no more than transmit the forces of shear in concrete into direct stress in the steel or direct stress in the steel into shear in the concrete. It is difficult to see how stirrups placed at intervals could perform this function.

From Fig. 1 it is seen that the maximum depth of beam is 20 times the diameter of rod. Hence the maximum ultimate bending moment is

$$354\ D^2 = 354 \times (20\ d)^2 = 141{,}600\ d^2$$

But the minimum span must be $200\ d$ to develop the adhesion in the rod. This is 10 times the depth, which agrees with the limiting span for shear. Since the shorter the span the greater the load per square foot for the same

resisting moment, we may obtain the limiting load per square foot thus: The moment in foot-pounds on a span $200\,d \div 12$ ft long is

$$\frac{w}{8}\left(\frac{200\,d}{12}\right)^2$$

where w is the load per sq. ft. Equating this to $141{,}600\,d^2$ we have $w = 4{,}080$.

This is the maximum ultimate load per sq. ft. that can be placed upon the top surface of a concrete-steel beam of stone concrete designed in the proportions here given and with the units here employed.

For cinder concrete, by the same course of reasoning, using 750 lbs. per sq. in. ultimate compression and 30 lbs. per sq. in. for adhesion and shear, we arrive at the following results:

Maximum ultimate bending moment $= 133\,D^2$.

Minimum span for adhesion $= 333\,d$, or $6.26\,D$.

Percentage of steel reinforcement $= .47$.

Minimum span for shear $= 6.65 \times$ depth.

Maximum ultimate load per square foot $= 3{,}910$ lbs.

It is recommended that for quiescent loads as in buildings a factor of safety of 3½ be used, and for rolling loads a factor of 4. The minimum span lengths, or, in other words, the maximum depths, should be used only in extreme cases. In special cases, where the beams would be clumsy, some of the rods may be curved up and anchored at the ends, thus making suspension rods receiving all of their stress at the ends. A rod thus placed in a curve would carry the shear corresponding to its own tension, leaving the concrete to take only that due to the transference of stress into the horizontal rods.

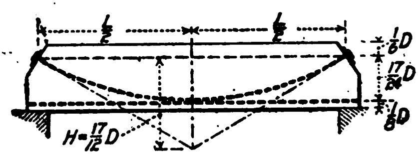


Fig. 2.

Fig. 2 shows the proportions for a beam of this design. The end detail of the rod should be capable of taking practically all of the stress on the rod; upsetting would not be necessary, however. Turning up rods at the end without anchoring the ends or making them continuous can scarcely be said to add any useful element of strength.

The shear carried by the curved rod will be found to be equal to the stress in the rod times $H \div \frac{1}{2} L$.

For slabs of short span the rules of design here given would demand close spacing of steel of small diameter. Steel mesh is the most suitable material, if the strands are straight; or if wires or rods are used, alternate strips of the slab may be considered as beams supporting strips of plain concrete between, which act as fillers, or which may be considered to perform the useful office of absorbing shock.

As an example of the application of the foregoing, given a floor with beams spaced 5 ft. and having a span of 20 ft. and these beams carried by girders of the same span; all to carry 100 lbs. per sq. ft. of live load, considered as quiescent. Assume a depth of slab of 4 ins. and 3 ins. spacing of rods. The rod will be ⅛ of the depth, or ½-in. from the bottom of the slab. The safe bending moment on the slab per foot of width, with the full area of the steel, would be $101 \times 16 = 1{,}616$ ft.-lbs. But the total moment, including 50 lbs. per sq. ft. for weight of slab is $150 \times 25 \div 8 = 469$ ft.-lbs. Hence in 3 ins. only $\frac{469}{1616}$ of the steel area is needed. Now, 1¼ per cent of $3 \times 4 = .15$ sq. in., and the fraction of this required is .044 sq. in. One-quarter-inch round rods will therefore suffice. One-two-hundredth of the span is .3 in. and the diameter of rods is less than this, as it should be. For the beam, assume a depth of 20 ins. This includes the depth of slab. Taking the dead load at 95 lbs. per sq. ft., the total moment is $5 \times 195 \times 20 \times 20 \div 8 =$ 48,750 ft.-lbs. The allowed moment on the beam per foot

of width is $101 \times 20^2 = 40{,}400$ ft.-lbs. The width of beam should therefore be 1.21 ft., or 15 ins. Rods will be 2 ins. from bottom of beam, and the area required is 1¼ per cent of 300 = 3.75 sq. ins. Three 1⅛-in. square rods will satisfy the conditions. The maximum diameter of rod allowed =

$$\frac{1}{200} \times 240 = 1.2 \text{ ins.}$$

For the girders, assume a depth of 32 ins., including the depth of slab. Using a dead load of 120 lbs. per sq. ft., the total maximum moment is $220 \times 20 \times 20^2 \div 8 = 220{,}000$ ft.-lbs. The allowed moment per foot width of girder is $101 \times 32^2 = 103{,}430$ ft.-lbs. The width of beam required is then 25½ ins. The area of rods required is 1¼ per cent of that of the beam, or 10.2 sq. ins. This is very nearly made up by six rods 1¼ ins. in diameter and two 1 5-16 ins. The allowed safe shear on the full section of the concrete girder is 2-3 of 200 (the nominal ultimate) ÷ 3½ (the factor of safety), or 40 lbs. per sq. in. The concrete will then take $40 \times 32 \times 25.5 = 32{,}640$ lbs. The total end shear is 44,000 lbs. As the concrete will take just about ¾ of the shear, we may turn up the two 1 5-16-in. rods and anchor them at the ends, as in Fig. 2. The stress in these rods will be the product of their area and 2-7 of 40,000 = 30,930 lbs. The shear that they will carry is found by the method given under Fig. 2 to be 11,680 lbs., which is just about the amount needed to supplement the shearing strength of the concrete. The diameter of the horizontal rods is just a trifle more than 1-200 of the span length, and the rods can be spaced a little more than π or 3.1416 times their diameter apart. The two curved rods may be placed between horizontal rods, as they give rise to no horizontal shear in the concrete; the latter merely hangs upon them as a saddle.

In a beam of the magnitude of the one just proportioned some saving of steel could be effected by placing the rods nearer to the bottom of the beam, or, say, 3 ins.

from the same, and making their area inversely proportional to the distance from center of steel to center of compression in concrete.

The writer believes that the foregoing principles of design, while they do not show the apparent economy of some special systems, would place concrete-steel designing in the class of sound engineering and force recognition where it is now decried.

Tests of beams thus systematically designed would show where any modification of units is needed.

THE DESIGN OF REINFORCED CONCRETE BEAMS AND SLABS.

Sir: I have read with great interest the article by Mr. Edward Godfrey on the design of reinforced concrete beams and slabs in your issue of March 15, and would say that he has handled the fundamental questions of the subject with great clearness and ability. He has, however, been intent on simplifying the calculations too much and thus fallen into the mistake of deriving empirical formulas. It seems to the writer that it is just as easy to err on one side as the other of mathematical complexity. The same path which has been pursued in deriving for steel beams working formulas which have stood the test of experiment and experience should be followed here. The fundamental assumptions, which are the solid foundations of the mathematical edifice, must be carefully weighed and made as broad and simple as possible. After having done this we can proceed with full confidence that we will not be led astray by a mathematical will-o'-the wisp. Where fanciful and complicated formulas have been derived, the fault is not to be found, as is often too hastily assumed, in mathematics as such, but in the lack of judgment or common sense in not starting aright. Mathematics can be compared to a powerful locomotive which, given a clear *track and a cool brain* to guide it, will arrive at its desti-

nation with accuracy and dispatch, but if put on a poor track and in the hands of an incompetent runner, will land in the wayside ditch.

The writer firmly believes that the mathematical engine has been started correctly by Mr. Paul Christophe in his classical work on reinforced concrete. The assumptions that he starts on are carefully weighed, simple and, if I may be allowed the expression, eminently sane. They are, with modifications proper to the new material, the same as those for the steel beam and should therefore lead to as satisfactory results. As a matter of fact, numerous experiments which have come to the writer's knowledge bear this out. Not to go into the subject too deeply at present, the writer would say that the percentage of steel for stone concrete of 1.25 per cent, which Mr. Godfrey gets, is excessive. Christophe demonstrates that for rectangular beams there is a certain percentage of steel with a corresponding position of neutral axis and depth of beam at which both the concrete and the steel are stressed up to their full respective values and which is therefore the economical one to use, as both above or below this point either the concrete or steel are over-stressed. Taking the depth of beams the same as the distance of the center of steel from the top of beam (which is the most rational thing to do, we can then add as much or as little concrete below as other considerations will dictate) we get for the percentage 0.60, for the position of neutral axis 0.385 of the depth h and for $h=0.108\sqrt{\frac{M}{e}}$ where M is the bending moment and e the width of beam. There are several other questions which the writer would like to take up at some other time. In conclusion he would say that he agrees with Mr. Godfrey that correct principles of design while they do not show the apparent economy of some special systems, would place concrete-steel designing in *the class of* sound engineering. There are enough *inherently weak points* in concrete-steel as regards work-

manship and materials that we should not add further uncertainties caused by strenuous inventors and adherents of special systems. The latter as a rule care very little for advancing the cause of good engineering as long as they can make it pay and cases are liable to arise where twisting and even falsification of facts may be resorted to in order to keep an otherwise discredited system in apparently good repute. Yours truly,

Henry Szlapka,

Resident Engineer, Toledo-Massillon Bridge Co.

342 Mint Arcade, Philadelphia, Pa., March 30, 1906.

Sir: I am in receipt of proof of a letter to you from Mr. Henry Szlapka, criticising my article on the design of reinforced concrete beams and slabs, published in your issue of March 15, and desire to thank you for this opportunity to reply to the same.

The first word that strikes me as strange in this letter is the word empirical, as applied to my formula. Empirical is defined as pertaining to or derived from experiment. In the sense that unit values, and sometimes final results, in formulas for strength must be derived from experiment, all such formulas might be classed under this head. But as commonly employed the term means a more or less scientific guess. The formula I derive is not empirical, for it is derived from the well-known theory of beams, using the strength of steel and concrete as determined from tests, and a position of the neutral axis determined from *tests to locate the neutral axis* (not by the uncertain method of using comparative moduli of elasticity). Mr. J. J. Harding, in a paper read before the Western Society of Engineers, Oct. 25, 1905, says, "Experimental methods are desirable for determining the position of the neutral axis, as it enables one to design a beam without making an assumption as to the modulus of elasticity of the concrete, which may easily vary 100 per cent."

For the *sake of* simplicity I have eliminated some of

the unnecessary kinks in the other formulas. For example, I have represented the stress in the concrete by a triangle and not by a triangle with a little segment added to the hypothenuse. The strength of concrete is not so well defined, even if conditions of manufacture and the materials appear to be identical, as to make it expedient to count in the almost insignificant segment of force with all of the complexity it entails in the formula, granting for the sake of the argument that the compression in the concrete does not vary as the distance from neutral axis. Suppose it should be discovered by use of exceedingly delicate instruments that steel under various stresses has a sliding modulus of elasticity, then the principle of the design of beams that the extension or shortening of the fibers varies directly as the stress would not be exactly correct, and the stress would not vary in intensity exactly as the distance from the neutral axis, but as the ordinates to some curve. Now there is no doubt at all that some mathematician would arise, with lots of time at his disposal, who would work out a general formula to take this variation into account for all shapes of beams. A difference in the strength of beams might be found amounting to one or more per cent. The strength of one bar of steel tested at two different places may vary several per cent, and the opinions of two engineers as to the proper factor of safety may vary still more; but this does not concern the mathematician, who has found a principle of mathematics violated by the commonly used "empirical" formula. Now I do not want to place a practical designer like Mr. Szlapka in a class with this mathematician, but I want to say that there is a lot of mathematical dust thrown into the eyes of designers who have not the time or the inclination to delve into these abstruse mathematical questions, but who would like at the same time to know a sound reason for using a formula, and to have that formula stripped of all elements

that complicate it without introducing any useful additional element of correctness.

As to $1\frac{1}{4}$ per cent of steel being too high, I quote from Prof. A. N. Talbot in Engineering News, July-December, 1904, page 125: "For 1:3:6 concrete, reinforcement as high as $1\frac{1}{2}$ per cent for steel of 33,000 lbs. per sq. in. elastic limit * * * may be used without developing the full compressive strength of the concrete." The proportions of concrete that I recommend are 1:2:4, a much stronger concrete, generally, than 1:3:6; it would allow a greater percentage of steel. Again, to quote Prof. W. Kendrick Hatt in "Engineering Record," Vol. 51, page 545: "In the writer's tests of beams under a center load, $2\frac{1}{2}$ per cent steel failed to develop the crushing strength of the concrete." In Engineering News of Feb. 15, 1906, p. 170, Mr. J. J. Harding states that for steel of an elastic limit of 35,000 lbs. he would use between 1 and $1\frac{1}{4}$ per cent of the area of the concrete. Mr. T. L. Condron, in a paper read before the Western Society of Engineers, March 15, 1905, says: "For extra strong concrete of about 1 of cement, 2 of sand and 4 of broken stone, the percentage of reinforcing may be increased to 1.25 per cent." (This in a discussion of 202 tests.) In view of the foregoing conclusions from the results of tests it is hard to see the force of Mr. Szlapka's bald assertion that $1\frac{1}{4}$ per cent is excessive.

Mr. Szlapka also objects to my assumption that the neutral axis is in the middle of the depth of concrete. As intimated, this was done on the strength of the results of tests or measurements to locate the neutral axis and not by means of a fancy formula, though it is not at variance with one of the most complex formulas I could find. In Engineering News, July-December, 1904, pp. 124-125, Prof. Talbot gives plottings of the position of the neutral axis. For 1.39 per cent reinforcement the distance from top of beam is almost exactly 50 per cent of the depth *from top of concrete* to steel. For 0.97 per cent it is about

0.42 of the same depth. Prof. Talbot gives this formula for finding the position: $k = 0.26 + 0.18\,p$, where k is the fraction of depth from top to neutral axis, and p is the percentage of steel. For 1¼ per cent this would be 0.485 of depth from top of concrete to steel. Prof. F. E. Turneaure in Engineering News, July-December, 1904, p. 215, says: "The diagrams show the neutral axis to lie at first very near the center of the concrete beam. As the cracks develop it moves gradually nearer to the compression side."

The use of the term "depth of beam" to designate the distance from top of concrete to steel is merely a matter of nomenclature. There are good practical reasons for using the outside depth of concrete as the "depth." In the first place it figures in the mind of the designer as the depth and governs the clearance, and it is usually in round numbers. In the second place a rule such as I give requiring ⅛ of the depth from center of steel to bottom of beam gives sufficient concrete below the steel to grip it and does not admit of skimping in this respect. It is a very essential point of design that the steel be surrounded with enough concrete to grip it effectually. This is a point often overlooked in designing.

It has not been my purpose to discredit mathematics but to point out the fallacy of unnecessary complication in formulas for use. In the struggle for existence the fittest formula has generally been found to be the simplest. Yours very truly,

Edward Godfrey.

Monongahela Bank Building, Pittsburg, Pa., April 12, 1906.

THE DESIGN OF REINFORCED CONCRETE BEAMS AND SLABS

Sir: The writer asks for the privilege of presenting a *reply to Mr. E.* Godfrey's letter in your issue of May 3, for *the sake* of continuing a discussion and sparring

for the last word, but because many points are raised of an exceedingly practical character, on which the writer earnestly seeks for light. If Mr. Godfrey's statement that 1¼ per cent is the proper amount of steel reinforcement is correct, then the writer is wrong by over 100 per cent and naturally feels great uneasiness over the safety of numerous structures which he has designed, and which have been built on his theory. His only justification must be, that he has erred, not through carelessness, but through unavoidable ignorance which is shared by a great number of his colleagues. He would like to ask Mr. Godfrey and beyond him, all practical designers who may notice this letter, whether they have used a percentage of 1¼ or one nearer 0.60 in their actual work. If they have used the former, they were more fortunate than the writer in not having had to meet close commercial competition.

A great number of times estimates made on the writer's theory have been beaten by reputable rivals and those of his designs which were executed did not differ by any material amount from other plans. The sizes given by his theory have usually compared closely with those by other more competent designers and, moreover, have passed the careful scrutiny of building inspectors. If he is radically wrong, he can only regret that he did not follow the advice of one of the best known bridge engineers given some years ago, who told him that he was too young to risk his reputation by dabbling with reinforced concrete, because in the course of time such structures would develop numerous flaws as yet hidden from observation. As a matter of fact, a concrete engineer has to take too many chances from poor materials and lack of care in execution of his work to add deliberately another grave risk of error from theory. Where so much uncertainty exists, he must accept the best current practice, select the most reasonable theory as a basis for his designs and await the consequences, assured that he cannot be held morally re-*sponsible for* defects which could not be prevented by using

all due precaution. The case would be different with a designer who use gross section instead of net of the steel reinforcement with the intention of making his customer believe he was getting more than he actually received. This is a matter in which ignorance cannot be excused.

Mr. Godfrey takes up considerable space in combating statements never made by the writer, which cover points on which, as a matter of fact, they are in agreement, as will be shown. He also talks about "mathematical dust" and "bald" statements. The "mathematical locomotive" naturally raises a great deal of dust in its flight through space like its material prototype, but that is a by-product which need not concern the man at the throttle. It can be left to be gathered up by the poor professional mathematicians, for whom most engineers profess such a marked contempt. As to baldness, the writer does not worry about being in that incipient condition himself, but feels very much concerned about his statements having reached that stage.

To bring matters to a definite issue the writer will briefly state the fundamental assumptions of Mr. Christophe's theory and ask Mr. Godfrey's and others' opinion, why in the present state of the art they are not the best, avoiding unnecessary refinement on the one hand, and being sufficient on the other to serve as a broad basis for elaboration. If they are correct, the writer feels no hesitation in following wherever they lead to. These assumptions are five in number: (1) Solidarity of concrete and reinforcement, provided the latter is so arranged as to assure a sufficient bond between the two; (2) invariability of plane sections; (3) invariability of the coefficient of elasticity of concrete in compression within the usual limits of stress; (4) neglecting the action of concrete in tension; (5) absence of initial stresses.

Mr. Godfrey speaks in all cases of ultimate values, whereas ordinary *theory* does not pretend to follow beyond *elastic limit*, either in steel or in concrete steel, and

it therefore seems that results deduced from testing beams to failure have little value as interpreted by our ordinary formulas. Moreover, beams of rectangular section, which are the only ones fully discussed in textbooks and experimented on, are very little used in actual practice as compared with beams of T section where the floor slab is counted on for part of the compression flange. What percentage of steel would Mr. Godfrey fix on for that case?

As to the writer's definition of "depths of beam" it is the only rational one to use, even if this statement is bald. Take a shallow beam, say 8 ins. deep and a deep one, say 36 ins., both within practical everyday limits. In the one case Mr. Godfrey would get 1 in. and in the other 4½ ins. for the amount of concrete under steel. Would he actually use these figures in a commercial design? This amount will rather be found to be a constant and that because of a very important consideration which Mr. Godfrey has not touched on. That is the protection afforded against fire, and here a great diversity of opinion exists. Prof. Chas. L. Norton, as quoted in Taylor & Thompson's book on concrete, considers 2 ins. in all beams and girders essential and most building regulations call for at least that amount. However, no more than 1½ ins. has been used very often and passed by some building departments. In a fire test of a system which has never used more than that amount it was incresed to 2½ ins., for the purpose of the test only, according to the writer's best knowledge and belief. There is also another point in this connection which has not been commented on, and which deserves careful consideration. In order to attach shafting or pipes to reinforced concrete beams, many different devices have been used, among others some in which a socket or similar device is in close contact with the reinforcement. In case of fire, heat is thus transmitted directly to the steel reinforcement with the result of heating it rapidly and causing the concrete protection to fall off, due to the unequal expansion, the concrete taking

a much longer time to heat to the same temperature than the steel; therefore, such devices should not be allowed, or, if they are, the buildings so constructed should not be held up as examples of fireproof construction. The last two points mentioned have apparently been frequently overlooked, but should not be slighted in the future.

If the inventor of a system is compelled to use features which cannot comply with scientific requirements and tries to pass them off under the sanction of the building department of a prominent city, he is no better than the packer of poisoned meat who uses the apparent sanction of United States government inspection which, when closely investigated, has no substantial meaning, but of which fact the public knows nothing.

As stated before, there are enough unavoidable uncertainties in the subject without adding other difficulties of our own making. In the rebuilding of San Francisco reinforced concrete will play an important part, but grave fear must be entertained that unscrupulous competition will result in much reckless and "skinned" work, unless held in check by regulations and inspection which are more than a mere name. Yours truly,

Henry Szlapka,,
Res. Engr. Toledo Massillon Bridge Co.
342 Mint Arcade, Phila., May 7, 1906.

The author did not reply to the above letter of Mr. Szlapka. The letter does not bring out much that is new. As to Christophe's assumptions, No. (5) would not hold in ordinary beams, as the shrinking of the concrete does put initial stress in the steel, and this disturbs all assumptions about the coefficients of elasticity, including No. (3), as made use of to locate the neutral axis. As to the amount of concrete that is proper to use below the steel, it would not be reasonable to use the same amount below a wire mesh in a slab as would be needed below and around a round or square bar 2 in. or so in diameter. The concrete is needed to grip as well as protect the steel,

and light sections of steel do not need as much concrete to grip them as heavy sections; also light sections do not need as much protection as heavy ones, because they are of less importance in the structure. One inch of concrete is sufficient in a shallow slab. It is totally inadequate in a large beam with heavy rods.—[AUTHOR.]

The Design of Reinforced Concrete Columns and Footings.

INTRODUCTION.—In the paper published in Engineering News of March 15, 1906, the writer derived some simple formulas for the design of reinforced concrete beams and slabs and some rules governing the proper proportioning of the steel for reinforcement and the limiting span length. Accepting the premises given in that paper, it becomes a simple matter to design beams and slabs in this comparatively new combination of materials. There seems to be no voice lifted up to deny the complete safety of beams thus designed to carry their loads, and no substantiated objection to the proposed method of design has yet been raised on the side of economy—the other end of the see-saw that the engineer must maintain in equilibrium.

Many so-called practical tests have been made on reinforced concrete construction in place that do not merit the name of tests. They are made on a small section of a floor supported on all sides by the contiguous construction, and do not test in any adequate sense the part of the floor immediately loaded. If a designer of steel construction should load a beam by placing a uniform load on a fraction of its length and declare that from the results of the "test" the beam is shown to be capable of carrying that uniform load throughout, he would be promptly ruled out; and yet this is the kind of test that is commonly held up as demonstrating the ability of some forms of concrete-steel design to carry enormous loads. Careful tests of isolated beams have been made in large numbers, whic

have proven with practical agreement the correctness of the premises and rules of proportion as well as the resulting formulas obtained by the writer in the paper above referred to.

On account of inquiries and criticisms that the writer has received from different sources, it is thought to be in place to amplify the former paper on beams and slabs by what immediately follows.

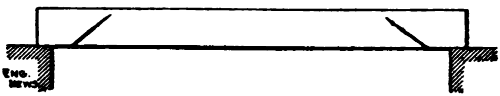


Fig. 1.

Tests have shown that rods of a greater diameter than about 1-200 of the span are apt to pull out of the concrete without breaking, on account of the lack of length in concrete to grip the metal effectually. Other tests have shown that beams having a greater depth than one-tenth of the span and containing horizontal rods at the bottom, or having rods turned up at the support and not anchored at the ends, will fail when loaded to destruction, approximately as per sketch Fig. 1. This demonstrates the inability of the concrete in beams so designed to carry the end shear, vertical and horizontal (which combine in the diagonal line in the figure), when the depth exceeds one-tenth of the span.

On the location of the neutral axis, theoretical methods of determining its position by the relative moduli of elasticity of steel and concrete are unsatisfactory, on account of the wide variation in the modulus of elasticity of concrete as determined by compression tests. There is, however, a more direct method of locating this axis, and one which gives more concordant results, namely, by measuring the relative extension and shortening of the lower and upper fibres in a beam under test. In a letter replying to a critic on this phase of the subject the writer gave the

following defense of his position, which is here repeated to make this paper complete for reference:

"In Engineering News, July-December, 1904, pp. 124-125, Prof. Talbot gives plottings of the position of the neutral axis. For 1.39 per cent reinforcement the distance from top of beam is almost exactly 50 per cent of the depth from top of concrete to steel. For 0.97 per cent it is about 0.42 of the same depth. Prof. Talbot gives this formula for finding the position: $k = 0.26 + 0.18\ p$, where k is the fraction of the depth from top to neutral axis and p is the percentage of steel. For 1¼ per cent this would be 0.485 of the depth from top of concrete to steel. Prof. F. E. Turneaure in Engineering News, July-December, 1904, p. 215, says: 'The diagrams show the neutral axis to lie at first very near the center of the concrete beam. As the cracks develop it moves gradually nearer to the compression side.'"

On the proper percentage of steel the writer's paper was also criticised by the person to whom the reply in the last paragraph was directed. This critic stated that 1¼ per cent is too large a percentage of steel. This criticism is correlative with the one on the location of the neutral axis. A higher location of that axis means a less percentage of steel, since it affords a larger lever arm or effective depth in the resisting moment. The writer's reply to this criticism was as follows: "As to 1¼ per cent being too high, I quote from Prof. A. N. Talbot, in Engineering News, July-December, 1904, page 125: 'For 1 : 3 : 6 concrete, reinforcement as high as 1½ per cent for steel of 33,000 lbs. per sq. in. elastic limit * * * may be used without developing the full compressive strength of the concrete.'" The proportions of concrete that I recommend are 1 : 2 : 4, a much stronger concrete, generally, than 1 : 3 : 6; it would allow a greater percentage of steel. Again, to quote Prof. W. Kendrick Hatt, in "Engineering Record," Vol. 51, page 545: "In the writer's tests of beams under a center load 2½ per cent of steel

failed to develop the crushing strength of the concrete." In Engineering News of Feb. 15, 1906, page 170, Mr. J. J. Harding states that for steel of an elastic limit of 35,000 lbs. he would use between 1 and 1¼ per cent of the area of the concrete. Mr. T. L. Condron, in a paper read before the Western Society of Engineers, March 15, 1905, says: "For extra strong concrete of about 1 of cement, 2 of sand and 4 of broken stone, the percentage of reinforcement may be increased to 1.25 per cent." (This in a discussion of 202 tests.)

The writer has been asked why he does not use the floor slab in connection with the beam as top flange, making a tee beam out of it. The buckle plate in a steel floor would not be used as top flange of the floor beams, though it may be secured firmly to the beams; and there is no good reason for using the concrete-steel floor slab as part of the beam. The slab is spread out too much for a proper distribution of the stresses, and it may have holes cut into it for pipes, etc., thus destroying its value as beam flange. Again, to include it in the calculations would add largely to the percentage of steel in the lower half of the rectangle of the beam. The rectangle of the concrete should not be called upon to take care of more than about 1¼ per cent of steel. Practically the only saving effected by counting in the slab is in the amount of concrete in the lower part of the beam. The concrete is needed there to protect the steel against corrosion and fire, and generally to give width enough to the beam to take care of the horizontal shear.

The subject of the limiting load that a concrete-steel beam of the proportions given in the former paper may carry may be amplified as follows: Assuming the neutral axis of a beam to remain in the middle of its depth for safe loads, it is seen that the safe resisting moment of the beam is directly proportional to the amount of steel. For steel in less amounts than 1¼ per cent the stress in the concrete is less than 2,000 lbs. per sq. in. at 40,000 lbs. per sq. in. on the steel, but cases may arise where it is

more economical to increase the relative amount of concrete. If the depth of a beam be doubled, its resisting moment per foot of width will be four times as great, assuming that the steel is still 1¼ per cent; that is, that the steel is doubled also. Now, if the amount of steel in the beam of double depth be divided by two, the resisting moment will be one-half as great. In other words, if the same rods be used in a beam of twice the depth, the resisting moment is doubled. The weight of the concrete and the depth of the beam are, of course, twice the values in the original beam, and these may be objectionable, when the double strength could be obtained by using a beam 1.4 times as deep, with 1¼ per cent of steel. However, where a large increase in depth and weight of concrete are not objectionable, the limiting ultimate load of 4,080 lbs. per sq. ft. on the top surface of the beam may be increased without necessitating the curving up and anchoring of some of the rods. Thus, if the limiting beam of one-tenth of the span be doubled in depth, with the same steel, it will have twice the strength and also twice the shearing area at the ends; hence it will have twice the ultimate capacity. In the foregoing the ratio in depth of one to two is taken to simplify the discussion. Any other relative

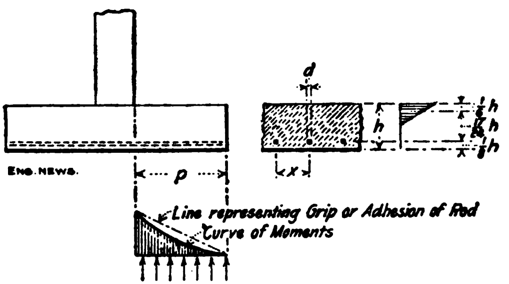


Fig. 2.

depths could be taken, and it would be found that starting with any given beam having 1¼ per cent of steel the

strength of a deeper beam of the same width with the same steel rods, placed 1/8 of the depth from the bottom, will be directly as the depths.

DESIGN OF COLUMN AND WALL FOOTINGS.—In proportioning concrete-steel footings the same principles of design hold; the beam, however, is a cantilever and not a simple beam.

Given a wall footing with a projection p and an upward pressure of the soil of S tons per square foot; in order to have shearing area at the edge of wall to take the upward pressure on the projection p, at 40 lbs. per sq. in. on the gross area, we have

$$40\ h=\frac{2{,}000\ S}{144}\times p,$$

$$\text{or } h = .35\ p\ S \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

That is, for every ton of soil pressure per square foot there must be a height h of 0.35 times p. Square rods seem to be the most suitable for footings. For the size of rod, in order to have 50 diameters of the rod in concrete at the point of maximum stress it is necessary to use rods of a diameter not greater than 1-50 of the projection p. The curve of moments and of stress in the rods is a parabola which lies at all points within the straight line representing the adhesion or gripping of the rod by the concrete. Hence p need be no greater than 50 d. Assume that

$$p = 50\ d \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

and that the stress on steel is 12,500 lbs. per sq. in. The amount of stress on the concrete will be the same as that on the steel, hence we may write for the allowed moment on the section x inches in width and h inches in depth,

$$M=12{,}500\ d^2\times\frac{17}{24}h=8{,}854\ d^2\ h.$$

But $h = .35\ p\ S$, whence $M = 3{,}099\ p\ d^2 S$ inch-lbs.

For the upward pressure of S tons per sq. ft. we have

$$M=\frac{2{,}000\ S\ x}{144}\times\frac{p^2}{2}=6.94\ S\ p^2\ x.$$

Equating these two values of M and using for p its value $50\,d$ we have

$$x = 8.93\,d, \text{ or say } 9\,d \ldots\ldots\ldots\ldots (3)$$

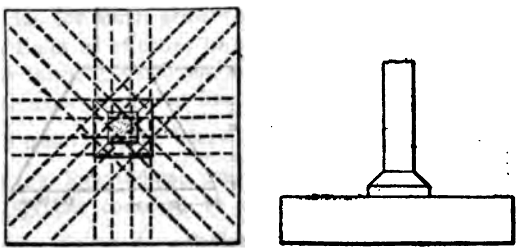
Hence for any upward pressure the same rods would be used, having a diameter 1-50 of p and spaced 9 times their diameter apart. Only the height h would vary for different earth pressures, as per equation (1).

The foregoing does not take into account the stress on the concrete, but we have seen that when the steel does not exceed 1¼ per cent of the concrete the stress in the concrete will not be excessive. When the steel in this footing = 1¼ per cent of the concrete,

$$h = 8.89\,d \text{ or } 8.89 \times \frac{p}{50} = .178\,p.$$

This would mean an earth pressure of only about a half a ton per square foot. Hence for all practicable earth pressures the amount of concrete is ample.

In square footings for columns the corners will have a projection 1.4 times that of the sides. The height h should be made 1.4 times that obtained by equation (1) using the projection at the side. Some rods should be laid diagonally, as shown in Fig. 3.



ENG. NEWS.

Fig. 8.

If cinder concrete be used in the wall footing, at 25 lbs. per sq. in. shear on gross area, we have

$$p = 83\,d \ldots\ldots\ldots\ldots (4)$$

$$h = .56\,S\,p \ldots\ldots\ldots\ldots (5)$$

$$x = 8.5\,d \ldots\ldots\ldots\ldots (6)$$

For a plain concrete footing in stone concrete, allowing 40 lbs. per sq. in. as safe modulus of transerve strength we have for the bending moment under edge of wall

$$\frac{2{,}000\ S}{144} \times \frac{p^2}{2}$$

and for the resisting moment $\frac{40\ h^2}{6}$, both on a rectangle h inches deep and one inch wide. Equating these we find the following to be very nearly true:

$$S\ p^2 = h^2 \ldots\ldots\ldots\ldots\ldots (7)$$

For a plain cinder concrete footing at 20 lbs. per sq. in. safe modulus of transverse strength, similarly we find

$$2S\ p^2 = h^2 \ldots\ldots\ldots\ldots\ldots (8)$$

By the foregoing we may obtain the relative cost of footings plain and reinforced. Thus at two tons per sq. ft. earth pressure, by eq. (1) $h = .7\ p$ (reinforced) and by eq. (7), $h = 1.4\ p$ (plain). By comparing the cost of the steel reinforcement with that of the additional excavation and concrete the relative costs of the plain and reinforced footings may be found.

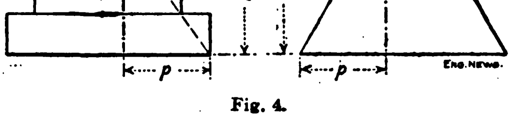


Fig. 4.

DESIGN OF COLUMNS.—In the design of concrete columns reinforced with steel it is essential to keep in mind the rational use of steel as a reinforcement in concrete, *namely*, to take tensile stresses. When a prism is *compressed* longitudinally, its diameter is increased, hence the *outer fibers* are put under annular tension. Hoops c

spirals bedded near the surface of a circular column will resist this tension and relieve the concrete. Hoops would have to be welded into solid rings to be of use, but a spiral may be used in long lengths. M. Considère in a series of tests on hooped concrete columns found that they exhibited great strength against compressive loads. They also showed regularity in the matter of failure. Where a plain concrete column would break suddenly without warning, a hooped column would hold together after cracks had shown partial failure to have occurred. These are desirable qualities in any kind of construction. Columns reinforced with longitudinal rods only did not show much, if any, advantage over plain columns. Longitudinal rods, however, are very useful in connection with a coil to space the loops and to tie the concrete together longitudinally, also to assist in resisting flexure in the column due to eccentric or horizontal loads.

It may be shown by a little computation that the area of metal required at a given unit stress to contain the liquid contents of a cylinder by resisting the bursting pressure is double that required to support the same load if the cylinder acts as a column. M. Considère found by the formulas for earth pressures that a shell filled with sand is 2.4 times as effective to sustain the load as it would be in the form of a column, at the same unit stress, and his experiments along that line confirm his calculations. This would mean that the lateral pressure of the sand is $\frac{1}{4.8}$ times its longitudinal pressure. If, then we treat the disintegrated concrete, at failure, as a substance whose pressure against the spirals is $\frac{1}{4.8}$ times that of a liquid confined in a cylinder, we may arrive at the tension on a spiral. In the tests above referred to some very high unit loads were shown at failure, while some of the tests failed between two and three thousand pounds per square inch. While the hoops or coils may hold the disintegrated concrete together and show high unit loads before ulti-

mate failure, the nature of the material as commercially made does not justify a higher safe unit load than about 550 lbs. per sq. in. The concrete between the spirals is in no better shape to resist compression than that in a beam. Using this unit load in short columns (under ten diameters in length) we have for our effective liquid pressure $550 \div 4.8 = 115$ lbs. per sq. in.

Let D = outside diameter of column and $\frac{1}{8} D$ = pitch of coil, in inches.

The tension on a coil =

$$115 \times \frac{D}{2} \times \frac{1}{8} D = 7.2 D^2.$$

Allowing 12,500 lbs. per sq. in. on the steel we have for square rods of diameter d inches

$$12{,}500\, d^2 = 7.2\, D^2.$$

Or, $$d = \frac{1}{42} D \text{ nearly.}$$

If the diameter of coil be made $\frac{7}{8}$ that of the column and that of the steel be made 1-40 of the same the stress on steel will be practically 12,500 lbs. per sq. in.

It is recommended that round or octagonal columns be used and that the full area of the circle be included as taking the load, also that square rods be used having a diameter one-fortieth that of the column in a coil with a pitch one-eighth that of the column. Where 1-40 D would give an odd figure the nearest sixteenth or eighth may be used and the pitch made five times as great.

In proportioning the longitudinal rods we may, in order to establish a rule for their size, follow the method employed by Marsh in "Reinforced Concrete" and allow them to take the outward force of the concrete between the spirals. If we assume eight square rods placed vertically inside of the coil, their clear span is

$$\frac{D}{8} - \frac{D}{40} = \frac{D}{10}.$$

The outward force from the 115 lbs. per sq. in. of assume

liquid pressure is $115 \times \pi D \div 8$ per inch. Being fixed ended their moment is

$$wl^2 \div 12 \text{ or } \frac{115\pi D}{8} \times \frac{D^2}{100} \times \frac{1}{12}.$$

Equating this to $12{,}500\ d'^3 \div 6$, the resisting moment of square rods of a diameter d', we obtain

$$d' = \frac{D}{38}.$$

This is close to one-fortieth of the column diameter; hence eight rods of the same diameter as that used in the coil may be placed on the inside of the coil and wired to the same. Where a coil ends the next should lap not less than half a coil, which would be about 55 diameters.

For columns more than ten diameters in length it is recommended that smaller unit loads be used, down to a minimum of 370 lbs. per sq. in. at 25 diameters. They should not be any more slender than one-twenty-fifth of the length. Between 10 and 25 diameters the allowed unit pressure would be found by the following formula:

$$p = 670 - 12\frac{l}{D}$$

where p = pressure per square inch,
l = length in inches,
D = diameter in inches.

It is recommended that the same reinforcement be used in all columns of a given diameter, so that flexure will be taken care of in long columns.

THE DESIGN OF CONCRETE STEEL BEAMS AND SLABS

Sir: Referring to the papers by Mr. Edward Godfrey, published in your issues of March 15 and July 12, I contend that the formulas and rules given by Mr. Godfrey are not sanctioned by practice.

Tests have shown that the adhesion between steel and *concrete decreases* with the diameter of the rods embedded

(Service Francais des Phares et Balises, etc), and following the rules given by Mr. Godfrey, to use rods of a diameter no more than 1-200 of the span of the beam, the designer will, in many cases, get too small rods. For example, for a beam of a span of 6 ft. the rods are to be

$$\frac{6\times 12}{200}=0.36\text{-in.}=\text{about } 3/8\text{-in.}$$

A good designer will never use 3/8-in. rods in a beam. Rods for beams especially should have sufficient stiffness not to bend at many points under the load of the concrete. If too small rods are used it will be very difficult to assure their distance from the bottom. To choose the proper diameter of the rods in each case the designer should have had practical experience; otherwise he may sometimes choose rods not easy to handle and which will not always allow him to get into the work the reinforcement made with a pencil on drawing paper in the office.

As for the space between the rods, the rule given by Mr. Godfrey will induce the designer to use too large beams. In such beams longitudinal cracks occur between the rods.

Stirrups are very useful and increase the strength of the beam. Well designed beams reinforced with stirrups will not fail as shown by Mr. Godfrey on the sketch on page 30 of Engineering News of July 12, though the height be greater than 1-10 of the span, and by the by, I would ask Mr. Godfrey why, in the example given by him (Eng. News, March 15), he does not follow his own theory, assuming for the main girder of a span of 20 ft. a depth of 32 ins. instead of 1-10 $\times$ 240 = 24 ins. As for the economy of beams designed according to the rules given by Mr. Godfrey it seems to me to be very problematical. A steel area of 1¼ per cent may be, when the depth and width of the beam are increased beyond certain limits, *too expensive* for the purpose. To get a good idea about *the cost of* reinforced beams I would suggest that the

weight of the steel be given in pounds and the amount of concrete in cu. ft. Very truly yours,

Michael Morssen.

38 West 26th St., New York City, July 15, 1906.

THE DESIGN OF REINFORCED CONCRETE BEAMS AND THE LOCATION OF MAXIMUM MOMENT IN A FOOTING

Sir: The writer has followed with much interest the development of a theory of reinforced concrete design as presented by Mr. Edward Godfrey, and the various comments and criticisms thereon by other engineers. That this is the writer's first participation in the discussion is due to the fact that he disagrees so entirely with Mr. Godfrey's fundamental assumptions that there is no possibility of carrying on a reasonable controversy. As stated in the columns of this paper, and elsewhere, the writer believes that reinforced concrete should be designed for the actual loads to be carried, the factor of safety being obtained by assigning allowable working stresses to the concrete and steel respectively. Where the distance of the neutral axis from the compression surface of a beam or slab is introduced in a formula, the value used, in the writer's opinion, should be that which exists under the working load, when the unit stresses in concrete and steel do not exceed their allowable limits. All authorities seem to agree that, even for the same beam or slab, the neutral axis is in an entirely different position under the breaking load. Without going into the question of formulas at all, there would seem to be no reason why reinforced concrete should not be subjected to the same rules as those which govern designing in other materials. These require the stresses under actual working load to be determinate, and that they shall not exceed the allowable values *prescribed* for each kind of material. No *design that has* not been made on this basis can comply

with building laws, architects' specifications, and the like, which almost invariably express their requirements in the form of allowable working stresses. It should be apparent that no one formula, involving such variable quantities as the coefficient of elasticity for concrete in compression, and the distance of neutral axis from compression surface, can or should be used for both ultimate and working loads. Let us say, for example, that a beam has been designed by such a formula to fail at four times the working load, and that the maximum compression in concrete at the breaking load is 2,000 lbs. per sq. in. Does anyone suppose that under the working load the maximum compression in the concrete is necessarily one-fourth of 2,000, or 500 lbs. per sq. in.? It may be more or less, according to circumstances, and yet how few adherents of this method re-compute their designs to ascertain the actual maximum working stresses, and to make sure they do not exceed the law or the specifications. Many would not know how to re-compute them, since their favorite formula would have to be altered by the substitution of a new, and higher, elastic coefficient for concrete, and a new, and greater, distance of the neutral axis from the compression surface.

In regard to the percentage of steel to be used, the writer has always maintained that this is a question not only of structural efficiency, but also of economy of cost and architectural restrictions and requirements. We know that in every member subjected to pure transverse bending, the total compressive stress on one side of the neutral axis must equal the total tensile stress on the other side. (Remarkable as it may seem, the writer has heard this well-known fact denied by men in very responsible positions.) There must, therefore, be a certain ratio of steel area in tension to concrete area in compression (neglecting the concrete in tension) which under working loads will give the maximum allowable intensities of stress in each *material. This* is the structurally economic ratio. If a *greater* percentage of steel be used, the maximum allowable

intensity of compressive stress in the concrete becomes the factor which fixes the resisting moment, and hence the working load. Under this same load, the intensity of tensile stress in the steel will not attain its allowable maximum working value, and the design will be structurally uneconomic. But for easily conceivable reasons the price of concrete may be very high, and that of steel very low, or again, architectural peculiarities may have limited the size or shape of the beam or slab. Under such conditions, the higher percentage of steel may give the greatest economy of cost, and the man who clings to a hard and fast percentage throughout his designing will be at a serious disadvantage.

The writer believes that concrete engineers will be almost unanimous in opposing Mr. Godfrey's statement that the slab, for a certain width on each side of the beam at any rate, should not be considered as part of the compression flange of the beam, if the concrete is deposited simultaneously and shearing stresses provided for in the design. The analogy, submitted by Mr. Godfrey in your issue of July 12, of a buckle plate in a steel floor does not seem to fit the case. If we may consider the table of a steel T-beam, whose width is generally more than ten times the thickness of the stem, as part of the beam, it would not seem unreasonable to consider a width of slab equal to ten times the width of the concrete beam, as an integral part of such beam.

From the foregoing remarks, it should be evident that the writer must forbear discussing Mr. Godfrey's mathematical deductions, since there is so great a difference of opinion at the very start. One point will be mentioned in regard to footing design. It is the writer's belief that where a cap stone or column base is superimposed on a footing slab, the maximum bending moment in the slab occurs at the center. To compute the slab as a cantilever whose length equals the projection beyond the cap stone, requires that the cap stone itself should be strong enough to take at its extreme edge the entire upward pressure on

the projection. As such is seldom the case, the writer believes in using the old reliable formula:

$$M=\frac{P}{8}(l-a)$$

for the bending moment at the center of the footing, P being the column load, l the length of footing and a the length of cap stone or column base. The reinforcing bars are arranged exactly as a grillage of iron beams would be, the strips running perpendicular to each other. In fact, a reinforced footing may be economically designed in practically the same manner as an I-beam grillage, the only difference being that the successive tiers are all in the same plane instead of being superimposed one above the other.

Before closing this letter, the writer wishes to acknowledge the fact that it does not refute Mr. Godfrey's theories. As previously stated, it is impossible to debate the matter from two such entirely different points of view, and like the closing addresses of two opposing lawyers before a jury, these letters mean but little until the verdict is rendered by the readers of Engineering News. Respectfully yours,

John Hawkesworth.

100 W. 80th St., New York, July 27, 1906.

Sir: I beg to thank you for the opportunity to reply to the letters of Mr. Michael Morssen and Mr. John Hawkesworth.

Mr. Morssen says that the formulas and rules given by me are not sanctioned by practice. My purpose in writing was largely to show that these rules in particular are overlooked in practice. Theory and practice must go hand in hand; each needs correction from the other. No system of construction was ever perfected by theory alone, and none was every perfected by practice alone. Practice *has* made many test-beams with short thick rods, and *these rods pull* out of the concrete just as theory, or the

rule given by me, would predict. This kind of practice is wasting steel that cannot develop its strength. Practice, of a construction in its infancy, is not a competent witness where alleged improvements on itself are concerned.

Mr. Morssen makes an indefinite assertion to the effect that the adhesion between steel and concrete decreases with the diameter of the rods embedded, and he cites French authority to back his statements, to which I have not access. No one will deny that the adhesion decreases with the diameter of the rods embedded. The tensile strength of the rods decreases with the square of the diameter of the rod embedded. Will Mr. Morssen assert that one rod 1-in. square and embedded 12½ ins. in concrete will hold with greater force than sixteen separate rods ¼-in. square embedded the same distance? The common standard of adhesion or grip of plain rods is the amount of area in contact with the concrete, and a simple calculation will show that rods of different diameters, in order to have the area in contact in proportion to their tensile strength will be embedded a given number of diameters. While it is admitted that the unit value of the adhesion is variable, as the other qualities of concrete are also variable, it is generally considered that a rod embedded 50 diameters in concrete will develop its full strength, or at least its elastic limit, if the rod be surrounded with a thickness of concrete equal to several times its diameter, the concrete having set in the air.

Mr. Morssen gives an example of a "beam" of 6-ft. span. If the span were only 6 ft., would not a slab be in order? He says a good designer will never use ⅜-in. rods in a beam. As I have used ⅜-in. rods in the design of a slab I will refrain from answering this.

I have no apology to make for the contention that the proper design of reinforced concrete demands that the steel be well distributed and of comparatively small diameter, rather than being concentrated in elements of large diameter.

Mr. Morssen says that the rule given by me will induce

the designer to use too large beams, and that in such beams longitudinal cracks occur between the rods. Can he cite any experience or results of tests to substantiate this?

Another dogmatic assertion of Mr. Morssen's is that well-designed beams reinforced with stirrups will not fail as shown by me though the height be greater than one-tenth of the span. I made tests on quite a number of beams with stirrups in a finished building. The principal mode of failure was just as my sketch shows. In Proceedings of the American Society for Testing Materials, Vol. IV, p. 498, Prof. F. E. Turneaure describes some test-beams which were 6 $\times$ 6 ins. and 60 ins. in span, reinforced with stirrups, spaced 3 ins. apart. On page 507 Prof. Turneaure says: "In but a few cases was the failure free from the influence of shearing stresses, the rupture usually occuring outside of the load and on a diagonal line." These beams were one-tenth of the span in height, and they had stirrups, and they failed about as my sketch shows.

The reason I did not "follow my own theory," as Mr. Morssen puts it in the girder of 20-ft. span is because it would give a clumsy beam to make its depth one-tenth of the span. Further, I wanted to give an example of a beam reinforced partially with rods curved up and anchored for their full stress at the ends, another part of the theory.

Mr. Morssen says that a steel area of 1¼ per cent may be too expensive for the purpose. Is it too much or too little? (requiring too much concrete). I have been criticised on the ground that it is too much from alleged theoretical considerations. If Mr. Morssen will look up practice as it is exhibited in descriptions of reinforced concrete construction in the engineering papers, I think he will find that leaving out the area of the slab, as having no place in the beam, 2 or 3 per cent of steel is not *uncommon.*

It is a pleasure to reply to a letter in the tone of that

of Mr. John Hawkesworth, which you have kindly submitted to me. This letter is from one who is manifestly a fellow seeker after the truth. A letter in a very different tone came to me recently, touching on some of the same points mentioned in this one, from one who has not the temerity to express publicly the final judgment which he snapped on my theory and deductions.

The question of using a certain unit stress on the steel and concrete and proportioning the beam on that basis, or of using certain ultimate values and then allowing as a safe load some fraction of the ultimate capacity is too extensive to be discussed here as a general question. When confined to a single case, as reinforced concrete, it is merely a question of means; identical results may be obtained by either means.

In reinforced concrete design it is convenient to use a certain factor of safety based on the ultimate strength of the parts because of the dissimilar materials dealt with and the desirability of having the same relative strength in each. In choosing an ultimate value for the steel I did not lose sight of the actual amount of the safe value. This is made clear by my first paper on the subject (Engineering News, March 15, 1906), for I there criticise the use of high elastic limit steel, because the resultant safe load on the steel when the factor of safety is applied is too great. Where is the difference between using 10,000 lbs. per sq. in. on the steel and 500 lbs. per sq. in. on the concrete and using 40,000 lbs. per sq. in. on the steel and 2,000 lbs. on the concrete with a factor of safety of four ?

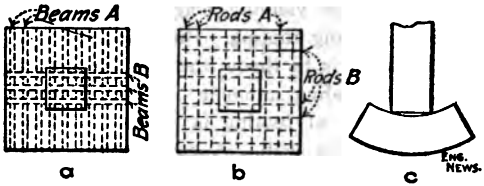
On the location of the neutral axis of a beam or slab I have said something before in these columns. The case is one that has entirely too many complications to be settled by theory with no other data than the relative moduli of elasticity of steel and concrete. The shrinking of concrete in setting is one of the complications. The tensile strength of the concrete is another. Measurements to *locate the neutral* axis of beams under test have shown

that it is close to the middle of the depth of the concrete beam under safe loads. I have taken it at the middle in all of my formulas. Now taking the neutral axis in the center of depth of beam and assuming that the intensity of stress in concrete varies uniformly from the neutral axis up there remains but one factor to fix the percentage of steel namely, the relative unit stress on concrete and steel If this is as 1 to 20 the percentage must be 1¼. This is very simply found by the principle which Mr. Hawkesworth states and which I have given in both of my papers namely, that the amounts of stress in steel and concrete must be equal. Any greater percentage of steel in a rectangular beam means a greater stress, relatively, on the concrete. Any less means a greater relative stress on the steel.

As to the floor slab acting as a T-beam: It would of course be allowable to use a part of the floor slab, provided it was placed at the same time as the beam, and provided the owner were forbidden to cut holes in the slab, as he might think he had a right to do. However, the T-beam method can only show to its credit a saving of concrete around the reinforcing rods, just where the concrete is needed to protect and grip the steel and to transfer the shear. I tested some beams in a building that were 3 ins. wide and were reinforced with 1⅛-in. square rods. Is this enough concrete to protect and grip a rod of this size or to take the horizontal shear? My tests conclusively proved otherwise.

Mr. Hawkesworth criticises my mathematical deductions on the wall footing, and states that where a cap stone or a column-base is superimposed on a footing slab, the maximum moment in the slab occurs at the center. A reinforced concrete footing would probably be surmounted by a concrete wall put in at the same time, and a column base would probably have a plinth of concrete upon which it rests, instead of being placed directly on the slab. In these cases the depth of beam would be augmented and the tension on steel diminished toward the center.

Mr. Hawkesworth gives an "old reliable" formula for the bending moment at the center of a footing. It is presumed that he does not mean that this formula expresses the bending moment at the center of a column, as this center is a vertical line and can have no section modulus. And yet this is what his reasoning seems to lead to. This is a formula for the bending moment in a row of I-beams in a grillage, at the center plane of the column or wall.



Beams A in the accompanying sketch, at (a), could be proportioned by this formula. Beams B could also be proportioned by the same. If, however, the latter spread out to the full width of the footing, those outside of the column base would be absolutely useless. There is no analogy between these steel grillage beams and the reinforcing bars as shown at (b). (I assume this is the manner of reinforcing referred to by Mr. Hawkesworth. My timid critic says this is the common way of placing the rods.) It is not even approximately correct to use the so-called old reliable formula. By that formula each set of beams takes the entire reaction and transmits it to the set above. If we assume that rods A in the reinforced concrete footing take all of the reaction, we must also assume that they give that reaction to such of rods B as lie under the column base. But this puts excessive load on those few rods, and leaves idle the rods B not under the column base. On the assumption that both sets of rods act together, necessarily more of the load must be taken *by the rods lying* under the base, and the formula is *seen to be inapplicable.*

By the use of diagonal rods the upward force on the footing at any part of the same is carried directly to the base by the shortest route. Each part is taken care of, and there is some excess of strength at the sides by reason of the additional depth used in column bases over wall footings.

The formula given by Mr. Hawkesworth, as applied to a wall, is based on the assumption that the wall load is perfectly uniform on the slab. This would be true if the wall were a fluid or yielding body that would take the shape of the deflecting footing. But when the footing deflects, the tendency is to throw the load close to the edge diminishing the bending moment. For all ordinary earth pressures the depth of footing is a large fraction of the projection beyond the wall. A considerable part of this projection could be made in a plain concrete footing assuming a low modulus of transverse strength. It is legitimate to make use of this surplus strength of the concrete to balance the small increase in the moment within the edge of the wall, which an only partially correct theory finds to exist. Yours very truly,

Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., Aug. 10, 1906

THE DESIGN OF REINFORCED BEAMS AND SLABS

Sir: I beg once more to use the columns of your paper to reply to the long and interesting letter of Mr. Godfrey published in your issue of August 23. In this letter Mr Godfrey repeats good and useful principles well known by all proper designers in reinforced concrete as well as by the writer. It is very old matter that theory and practice must go hand in hand, that rods for reinforcement should be embedded a given number of diameters to develop their full strength and that the proper design of a slab and beam demands that the steel be well d

tributed and in small rods rather than being concentrated in elements of large diameters. But repeating these principles Mr. Godfrey does not say that he maintains again his rules in their full conclusion as they are printed in your issues and with which the writer does not agree. These principles are (issue of March 15, pp. 291, 292):

(1) The rod should be no more than 1-200 of the span.

(2) The maximum depth of a beam is 20 times the diameter of the rod embedded or 1-10 of the span. The maximum depth should be used only in extreme cases.

In your issue of July 12, p. 30, Mr. Godfrey says that by his rules it becomes a simple matter to design beams and slabs in this comparatively new combination of material, which is to say that design will be much easier than it was till now. This is not the writer's belief. The examples given by the writer in his first letter (August 2) were only to show to what design these rules may lead if the designer follow them literally. Good designers who know their business will in the most cases not follow these rules as their own author has done in his example cited. For poor designers who overlook the fundamental principles stated above and who do not know much about reinforced concrete, rules as these given by Mr. Godfrey become a danger and make them advance from bad to worse. Just because, as says Mr. Godfrey himself, this kind of construction is in its infancy, the writer's belief is that it will be a guess matter to give now rules to govern the proper proportioning of the steel and the limiting length of the span, etc. Nevertheless, papers as these of Mr. Godfrey and very useful and interesting.

One point which seems to me to be overlooked by Mr. Godfrey is the fact that reinforced slabs and beams are in the most cases designed as continuous elements and the rods of one slab or beam overlap the rods of the next, and then the length of the rod embedded in concrete exceeds in many cases the length required by the calculation *though the diameter be larger than 1-200 of the span.*

Mr. Godfrey criticises the writer's example with a beam of a span of 6 ft., saying that a slab should be in order for this span. I would answer that beams at a span of 6 ft. or less are often to be designed over openings and have sometimes to carry large concentrated loads which a slab would not do.

As to my so-called (by Mr. Godfrey) dogmatic assertions about the failure of well designed beams, overlooking laboratory tests with small elements, I would refer Mr. Godfrey to the text-books of Christophe, p. 490 (French text) and of Prof. Mörsch, "Der Betoneisenbau," p. 121, etc., where the failure of beams is discussed, and he will see that it is not only my own judgment. The finished building test by Mr. Godfrey in which beams have failed by shear under the test load (generally 1½ to 2 times the live load for which the floors are designed) should be of a very poor design and for this reason cannot be cited as reference. The writer has had the opportunity to assist in tests made in France (Paris) and in Germany (Charlottenburg, near Berlin), and he himself has made many tests on large elements and got other results than those cited by Mr. Godfrey. The writer believes that deep beams designed only for the bending moment will fail by shear as well as beams of a small height will do when they have to carry a concentrated load near the support and the reinforcement was not designed for the purpose.

To substantiate the assertion that in large beams longitudinal cracks occur in the spaces between the rods, the writer would say that an analogous fact occurs in slabs reinforced in one way and to avoid these cracks additional cross-rods are embedded. In some large beams the writer has himself verified these cracks in cases in which special arrangements, as little cross-bars or special U-shaped stirrups going under all rods were not provided.

As to the steel area to be used the writer knows that *sometimes* beams are designed with a steel area of 2 per *cent or more* and his idea was not so much to criticise

this amount, as to give a suggestion that when speaking about the cost the amount of concrete be given in cubic feet and the steel in pounds per lineal foot of beam or per square foot of slab. Very truly yours,

M. Morssen.

38 West 26th St., New York, N. Y., Aug. 26, 1906.

The author did not reply to the above from Mr. Morssen. Of course if Mr. Morssen does not elect to accept simplified assumptions, he does not come in the class to whom the design of reinforced concrete beams is made a simple matter by those assumptions. Mr. Morssen begs the question in citing continuous beams, where the rod passes into the next beams. There are many beams made that are not continuous, and, in any event, there must be an end to a line of continuous beams. Further, a short beam over a six-foot opening supporting a heavy concentration would probably be isolated. If such a beam had thick rods not positively anchored at the ends, it would be faulty, and the rods would pull out before they received their full stress, for the simple reason that they would not be bedded deep enough in the concrete to have the necessary grip.

It is a far cry from a slab not reinforced transversely to a beam of width enough to keep the steel reinforcing rods well separated. An isolated slab would not have the tendency to crack longitudinally that there exists in a slab built in a structure. So a beam of good width would not have shrinking forces on each side of it tending to split it longitudinally.—[AUTHOR.]

CONCERNING THE STRESSES DUE TO SHRINKAGE IN REINFORCED CONCRETE

Sir: In an article in Engineering News of March 15, 1906, on "The Design of Concrete-Steel Beams and Slabs," by *Edward Godfrey,* he says (first column, page 291):

The fact that concrete in which steel is embedded has

been stretched out in tests without cracking to elongations that would rupture plain concrete is evidence that the concrete in setting has shrunk, thus putting the steel under an initial compression which must be overcome before any stretch occurs in the concrete.

Now it seems to me that when the steel is put under compression by shrinking of the concrete, the latter is at the same time put under tension, and any further loading of the beam would all be carried by the concrete, so that it would have to take all the tension until it has stretched so far that all the initial compression has been taken off the steel.

Edwin Squire.

Claremont, California, July 19, 1906.

The author did not make reply to the above. He sees the force of Mr. Squire's argument. He is, however, still of the opinion that the shrinking of the concrete in setting has much to do with its integrity under stress. The tests which have been heralded so widely in this country as disproving Considère's conclusion, that concrete with steel embedded will withstand cracking when the calculated stress in the steel is large, were made on beams that were kept in water and thus prevented from shrinking. (These tests were made by Prof. Turneaure. See Proc. Am. Soc. for Testing Materials, Vol. IV.) It is possible that the concrete, in the beam which sets in air and contracts, takes more than its share of the stress, so that the tension in the steel, and the consequent stretch, are not so much as calculations show. It is further possible that between a unit compression x in a steel rod and a unit tension y the theory may not hold in its exactness that the difference in length is the same as that which would be produced by a unit tension of x plus y on an unstressed rod. In any event practice and tests show that calculated stresses in the steel in air dried beams up to 10 or 12,000 *lb. per sq. in.* do not produce perceptible cracks. Excessive *shrinking* is, of course, harmful.—[AUTHOR.]

ON PROPORTIONING THE REINFORCEMENT OF CONCRETE COLUMNS

Sir: In your issue of July 12, 1906, Mr. Edward Godfrey treats at some length the question of design of reinforced concrete columns, and develops formulas for proportioning the reinforcement based upon the theory that the concrete at failure becomes a granular mass, whose lateral pressure is 1 divided by 4.8 times that of a liquid confined in a cylinder. He arrives at the conclusion that in a circular column of diameter *D* the spiral reinforcement should be a square rod, whose side is equal to 1-40 *D*, the pitch of the coil equal to ⅛ *D*, and the longitudinal reinforcement 8 square rods of the same dimensions as the spiral.

It seems to the writer that your correspondent in his fundamental assumptions and deductions therefrom has been taking a "long shot."

The relation between the lateral and the longitudinal pressure of a granular mass confined in a cylinder and subjected to its own weight or to additional pressure, bears very little resemblance to that of a fluid under similar conditions, for the following reasons: In treating a confined granular mass we are dealing with a substance in which there is an appreciable friction between the particles of its own mass, as well as a friction between these and the walls of the confining cylinder. This is obvious, for one has only to consider that a bucket full of sand thrown out upon a board does not spread out to a uniform depth, but remains in a more or less cone-shaped pile, neither will it slide from the board until the latter is tilted to a considerable angle. Owing to these physical facts, when a granular mass is confined in a cylinder its weight is supported partially by the bottom of the cylinder and partially by the sides acting as a column, while the sides are at the same time under tension due to the lateral pressure.

This action of granular masses confined in bins, and

the relation of lateral to longitudinal pressure, have been very thoroughly investigated by Mr. J. A. Jamieson, of Montreal, and made the subject of a very complete paper presented before the Canadian Society of Civil Engineers, and later published by Engineering News (March 10, 1904). Mr. Jamieson found, for such granular masses as wheat, corn, flaxseed, etc., that when no settlement of the bin walls occurred, approximately only 20 per cent of the confined mass was supported by the bottom of the bin, while the remaining 80 per cent was supported by the sides acting as a column; that the lateral (maximum) pressure at any point was practically constant and equal to 6-10 of the vertical pressure when the height of the grain column was equal to or exceeded about 4 times the diameter of the base; also that it made little or no difference in these relations if the bin was round or square. Among many experiments recorded by Mr. Jamieson was one on dry sand, and it is interesting to note that he found approximately the same results as for wheat; that is, a ratio of lateral to vertical pressure of 0.65, and 81.5 per cent of the total weight carried by the sides of the bin.

A very simple experiment in line with the above may be performed by making a tube, say 10 ins. long and 2 ins. in diameter, from thin writing paper, placing it on end and filling it with dry sand. When the sand within is subjected to pressure, the first noticeable feature is a buckling of the walls of the tube, generally about 1-3 the height up from the bottom. Additional pressure causes rupture, the resulting tear running parallel to the long axis of the tube. If the length of the tube be made 20 ins., the diameter remaining the same, and the sand subjected to pressure, there is a buckling of the walls of the tube followed by a bending of the whole and a failure near the center, the tear in this case being at right angles to the long axis. In the first instance the walls acted as a column until they failed (buckled), then ruptured under a tensile stress such as would be caused by hydraulic

pressure. In the second instance the walls failed by buckling, then ruptured under the stress caused by the bending moment at the center.

Sand confined in a cylinder can be made to withstand enormous loads, limited only by the strength of the confining cylinder. This is the principle of sand wedges used in arch construction, and is well known.

If we are going to treat the reinforced concrete column at rupture as so much disintegrated matter or as sand, and from this assumption develop formulas for ascertaining the dimension of steel spirals to withstand this disintegrating action, then there are more and different factors to be considered than those used by Mr. Godfrey. In view of Mr. Jamieson's experiments, quoted above, the lateral pressure would be 0.60 of the longitudinal and not 0.21 (i. e. $1 \div 4.8$) as stated by your correspondent. That is, the lateral pressure is about three times as great as he assumes. The matter of the walls (spirals) carrying 80 per cent of the loading as a column is not taken into consideration by him at all, and it seems to the writer that it is radically wrong to apply to a concrete column this theory of disintegration which logically demands that the walls (or longitudinal reinforcement) take 80 per cent of the imposed loading. To follow up this line of reasoning must induce one to dispense with the concrete altogether and employ an all-steel column.

A mild steel rod in the form of a coil is metal not well disposed to act as a column carrying loads, and experiments on reinforced columns in which the reinforcing is only spirals or hoops show a much larger degree of compressibility under loading than columns in which longitudinal rods of considerable size are employed. This is well brought out in Mr. Howard's article in the issue of Engineering News for July, 1906, in which he says, "Hooped columns are a distinct group and decidedly more compressible than the others." In the same article it is *clearly shown that* hooping a column of 1-2-4 concrete *will add greatly to* its ultimate strength, and so will the

addition of a fair amount of longitudinal metal (2.86 per cent), while a combination of the two will further increase the ultimate resistance under loading. A mixture rich in cement and unreinforced (1 of cement to 1 of sand) is 20 per cent stronger than a 1-2-4 concrete reinforced with 25 hoops and 4 angles, and considerably more rigid under loading. In these tests on 1-2-4 rock concrete the hoops were 1½ ins. wide by ⅛-in. (given as 0.12 in.) thick spaced at various intervals. The smallest recorded number (13) increased the strength of the column 58 per cent above that of the plain unreinforced piece, while the largest number (47) increased the strength 274 per cent, raising it to 5,289 lbs. per sq. in. In the latter case the spacing of the hoops was a trifle over 2½ ins., or the neat opening between the hoops was 0.66 times the width of one hoop. Mr. Godfrey's deductions that the diameter of the square spiral rod should be 1-40 *D*, spaced ⅛ *D*, leaves a clear opening of 1-10 *D* between the rods, which opening is four times the width of the rod. This does not look like metal well placed to confine a disintegrating mass.

In a reinforced beam we place the metal where it will resist the tensile stresses, leaving the plain, unreinforced concrete to take care of the compressive stresses of 500 lbs. per sq. in. or greater. It this is good practice (and it is a common one) why should we design a column on a basis of 550 lbs. per sq. in. of compressive stress and then reinforce it. In other words, unless we either figure upon an allowable unit of compressive stress of much greater amount than 550 lbs., or assume that the longitudinal reinforcement carries a portion of the imposed load, then we have not reduced the size of the column under that of one not reinforced, and have added largely to its cost. It is the large size of columns (compared with steel) that in actual practice makes them so objectionable to many architects. This size can be reduced to reasonable limits by using a concrete mixture rich in cement, reinforced with spirals or hoops that form a close ng mesh and longitudinals to which they are firmly

attached, assigning to the concrete a high compressive unit of resistance and to the longitudinals a portion of the imposed load, but not by the formulae and method suggested by your correspondent. Very truly yours,

G. B. Ashcroft,
C. E. Assoc. M. Can. Soc. C. E.
Supt. Roman Stone Co., Ltd.

Toronto, Ont., Aug. 17, 1906.

THE DESIGN AND THE BEHAVIOR OF REINFORCED CONCRETE COLUMNS

Sir: I have before me proof of Mr. G. B. Ashcroft's letter in which he objects to my method of proportioning a reinforced concrete column, and thank you for the opportunity to reply to the same. The answers to Mr. Ashcroft's objections are found partially in the papers which he cites and partially in his own letter.

I have always considered the paper by Mr. J. A. Jamieson on tests on grain bins as a very valuable contribution to engineering knowledge. These tests were made to determine the relation between the head of grain in a bin and the horizontal and vertical pressures in the grain, and were not made with a view of finding the bursting pressure of a confining cylinder which has no longitudinal strength. One experiment made by Mr. Jamieson on a cloth cylinder showed what one naturally would suppose, namely, that if the walls can yield vertically the grain itself will support all of the weight. In a steel cylinder a large part of the weight would be carried by the shell. A coil in a reinforced concrete column would act like the cloth cylinder; that is, it would take none of the load on the column. Unfortunately for column investigators, Mr. Jamieson did not measure the lateral pressure on the cloth cylinder. However, among his experiments there is one which has a direct bearing on this subject. The experiment referred to is one in which

bin having sides that could be raised and lowered was filled. The pressures on the bottom and sides were measured, and the latter was found to be about six-tenths of the former, as stated by Mr. Ashcroft. Upon lowering the sides all of the grain as well as the sides themselves was supported on the bottom. Mr. Jamieson says (Eng. News, March 10, 1904, p. 238):

"On the bin being again lowered to its original position, while no increase of lateral pressure was shown by the side diaphragm, there was a very large increase of pressure on the bottom diaphragm, or sufficient to cause the water to flow out of the top of the 4-ft. gage glass tube, which was not, therefore, long enough to record the pressure; in fact, the total weight of the grain was then resting on the bottom diaphragm, and in addition the grain was acting as a column to support the weight of the bin itself."

Now the lateral pressure in this experiment at the bottom of a bin 78 ins, high was .1894 lb. per sq. in. This is only one-twelfth of the vertical pressure instead of being six-tenths.

In one of the experiments referred to by Mr. Ashcroft, among those made at Watertown Arsenal, is one on a column reinforced with wide bands. By my count these bands comes within a half inch of each other instead of an inch. This column stood 5,289 lbs. per sq. in. It is safe to say that the concrete was in a state bordering on disintegration under this load. The lateral pressure on the bands could not have exceeded a force that would stress the steel to its ultimate strength, and this will give us a basis for determining that pressure. In the 2 ins. of a 10-in. column occupied by one band an equivalent fluid pressure would be $2 \times 5 \times 5{,}289 = 52{,}890$ lbs. If the net area of the band were an eighth of a square inch the unit tension on the basis of a liquid pressure would be 423,000 lbs. I do not know what grade of steel was used, but if it were good for 80,000 lbs. per sq. in. t

lateral pressure could not have been as much as one-fifth of the longitudinal pressure.

Mr. Ashcroft's advocacy of the closer spacing of rings and of wide rings leads to the conclusion that the best column is a steel tube filled with concrete. There is no doubt that this would make a strong column, but it is a composite column, with all that the term implies in the way of uncertainty in the distribution of stress, and not a reinforced concrete column. There are objections to such a column for ordinary use that it is needless to mention.

The objection to a flat bar in reinforced concrete is that the holding power of concrete is due to its gripping the steel, rather than mere adhesion. The concrete tends to shrink away from the side of a broad flat bar, whereas it grips firmly a small square or round bar. Further, the concrete on the outside of a broad flat bar near the surface may be easily knocked off. Some years ago I observed some reinforced concrete beams in which the reinforcement was a number of flats, which were brought up and hooked over the flanges of steel beams. The concrete below the bars fell off in large chunks when the forms were removed.

The objection to longitudinal steel in a column, that is, to the counting upon it as taking part of the column load, is that this, too, makes the column a composite structure and not a reinforced concrete column.

I cannot see any objection to lack of rigidity, as it is called, in a column, that is, to a column that will shorten a proportionately large amount under load. Wooden columns with a safe load of 1,000 lbs. per sq. in, and $E = 1,000,000$ will shorten three times as much as steel columns at 10,000 lbs. per sq. in. $E = 30,000,000$, but wooden columns are not objectionable merely on this account. In a beam rigidity is very essential, as the deflection can be felt or readily measured; but a little *additional deflection in* a column cannot be detected with-*out the most careful* measurement. An explanation of

the lack of rigidity in reinforced columns as observed in the Watertown Arsenal tests is found in the tendency of concrete to shrink in setting. It does not seem to me to be necessary to assume that fissures occur, on this theory, as suggested by Mr. Howard (in Eng. News, July 5, 1906). The tendency to shrink may be there and may be offset partially by the steel embedded. The concrete is then like a spring which is held from contracting quite down to its normal length by the steel. Again the tendency to shrink away from coils or hoops would make a swelling out of the column (due to the load) necessary before the coil is brought into action.

Mr. Ashcroft says, "In a reinforced beam we place the metal where it will resist the tensile stresses, leaving the plain unreinforced concrete to take care of the compressive stresses of 500 lbs. per sq. in. or greater." This is exactly what we ought to do with columns. A short block one or two diameters in height could safely be loaded to 500 lbs. per sq. in., but a plain concrete column, if loaded to this amount, is apt to break suddenly by bulging or flexure. The coils and longitudinal rods are used to overcome this weakness, just as the reinforcing rods in a beam are used to overcome the weakness of the beam in tension. The strength of the concrete in either case is practically that of concrete in short blocks.

Confined in a cylinder, concrete (or even loose sand) has a very great power for carrying loads; but while this fact is very useful in some lines it has little bearing on reinforced concrete design.

Has Mr. Ashcroft's experience with sand jacks shown him that the sand issues from the gate with a pressure approaching six-tenths of the unit load upon it? Would he design the walls of the jack for this pressure? Yours very truly,

Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., Aug. 31, 1906.

[*In connection* with the explanation, above offered, of *compressibility* of concrete columns in certain cases,

the old fallacy that two springs combined in opposition are more sensitive than a single spring recurs to mind. Really, the stiffness of two springs, whether in opposition or in parallel, is the sum of their individual rigidities, as a diagram will readily show. Now, applying this to the reinforced concrete, we have the concrete in tension due to shrinking, and the steel in compression to an equal amount. The rigidity of the combination, that is, the load required for unit compression, is just as great under these circumstances as if both steel and concrete were initially unstressed. The only case in which there is an exception to this rule is when there is a certain range of inelastic motion, or when the elastic system is changed at a given period; the former is instanced by local crushing at the bearing surfaces, the latter by loose fit of one of the elements which operates to keep it free from load until a certain compression is reached.—Ed.]

A FURTHER NOTE ON THE ANALYSIS OF REINFORCED CONCRETE COLUMNS

Sir: Referring to your comment on my suggested explanation of the lack of rigidity in a reinforced concrete column, in Engineering News of Sept. 6, I beg to suggest further that the concrete is not in all respects like a spring, but is restrained in the neighborhood of the steel more than at other points. Suppose, for example, that a column of concrete with a central longitudinal rod be molded. In setting, the concrete will tend to shrink, and near the outer edge of the column this shrinkage will actually occur, in practically full amount. But near the steel the shrinkage will be counteracted by the resistance of the steel to compression, and only a small part of the normal shrinkage will take place, resulting in longitudinal tension in this zone of concrete, and longitudinal compression in the steel. The end faces of the column *will, therefore, not* be planes, but will be low at the per-

iphery, while the end face of the steel rod will be the highest point. Hence, when the column is put in the testing machine for a compression test, the load on the end blocks will be received by the steel and the immediately surrounding concrete before the concrete away from the steel receives its load. The concrete near the steel would therefore be under greater unit stress, and the apparent rigidity might be less than plain concrete. The case of a spring was cited to show that it is not necessary to assume that fissures exist where the concrete is not shrunk down to its normal size. Yours very truly,

Edward Godfrey.

Pittsburg, Pa., Sept. 7, 1906.

The Designing of Reinforced Concrete Retaining Walls.

The lateral pressure of earth is too uncertain to be defined by any law, and hence a correct theory for proportioning a retaining wall in the general sense of the term is not possible. Some earths will remain vertical for some time without any confining structure; some will even stand tunneling without caving in. A board fence will retain a considerable height of earth laid against it for awhile. On the other hand, slips are liable to occur in earth after it has stood for some time unsupported. These often exert great force.

The lateral force of grain is, no doubt, nearly constant for a given kind of grain under given conditions. The same is probably true of sand, if one of the conditions be a definite proportion, or the entire absence, of moisture. But in the case of the substances generally classed under the term earth there is, as intimated, a variety of states, ranging from soft mud that will exert a lateral pressure approximating a fluid pressure, to shales that exert no active lateral pressure. A condition approximating a fluid pressure ought to be avoided in any

good design by proper drainage. Two cases calling for special exercise of judgment are (1) the one mentioned, of shale, which is itself a sort of retaining wall of little durability, and (2) a case where large slips are probable, either due to an inclination of the under lying rock toward the retaining wall, or to a heavy surcharge, that is, earth sloping steeply above the top of the wall.

This paper treats of a wall to retain ordinary fill or prevent natural earth from slipping. The most severe test of a retaining wall is usually the freezing and thawing of the earth around it. The forces produced thereby are quite indeterminate. They can, however, be largely diminished by drainage. It has been found that masonry walls having a base one-third the height of the earth retained will resist these forces and retain any ordinary earth with complete rigidity. That this relation between the base and height agrees with the theory of earth pressures commonly employed is shown by the following.

The theory referred to is based on deductions from the assumed action of a granular mass having no cohesion between its particles. It is found that the effect is that of a wedge of the material sliding without friction on a plane the slope of which bisects the angle of repose of the material and the vertical back of the retaining wall. It is customary in discussions on retaining walls to treat the force due to this wedge as though it were produced by a solid block with its center of pressure at a distance of one-third of h from the base. The calculations are simplified by treating it as a liquid pressure, the weight of the liquid per cubic foot being a certain fraction of the actual weight of the earth. This fraction is equal to the square of the tangent of the angle $\frac{a}{2}$ (Fig. 1), a being the complement of the angle of repose.

The *angle of repose* of ordinary earth is about 45°, *half of its complement* is 22½°, and the square of the

tangent of 22½° is .1716, or nearly one-sixth. Hence we use the pressure of a liquid having a density on sixth that of earth we will have the effect of this later pressure so far as horizontal forces are concerned. Th will neglect the vertical component of the force of th sliding block. A small and uncertain amount of th vertical component would act in friction on the ba of the wall; the remainder would be carried on the i clined surface upon which the block slides. As th force of friction tends to give greater stability to th wall, it is on the side of safety to neglect it.

Fig. 1. Diagram of Earth Pressure Against a Retaining Wall with Vertical Face.

Now from Fig. 1, if we take moments around X an use as the weight of earth 100 lbs. per cu. ft., and masonry 150 lbs. per cu. ft., we have for stability th following equation of moments:

$$150\,h\,b \times \frac{b}{6} = \frac{100\,h}{6} \times \frac{h}{2} \times \frac{h}{3},$$

from which we find

$$h = 3\ \ b.$$

Hence, using only the horizontal forces and neglectin adhesion or friction on the back of the wall, we fin that a wall having a height three times its base, whic is usually considered in good proportion for a retainin wall, will be found to be stable against ordinary earth.

In retaining walls of reinforced concrete the eart itself may be utilized to prevent the overturning of th

wall, not by its uncertain friction or adhesion but by its weight.

A form of reinforced concrete retaining wall coming into use is composed of a front curtain wall and a bottom slab, both reinforced with horizontal rods; wall and slab are united by ribs at intervals. Fig. 2 is a modification of that style of construction, by the addition of steps at the junction of the front wall and the bottom slab. The purpose of these steps will be made evident further on. The ribs are spaced a fixed distance apart, the same for all heights of wall. The design is to be used only for walls having a base width of 5 ft. or

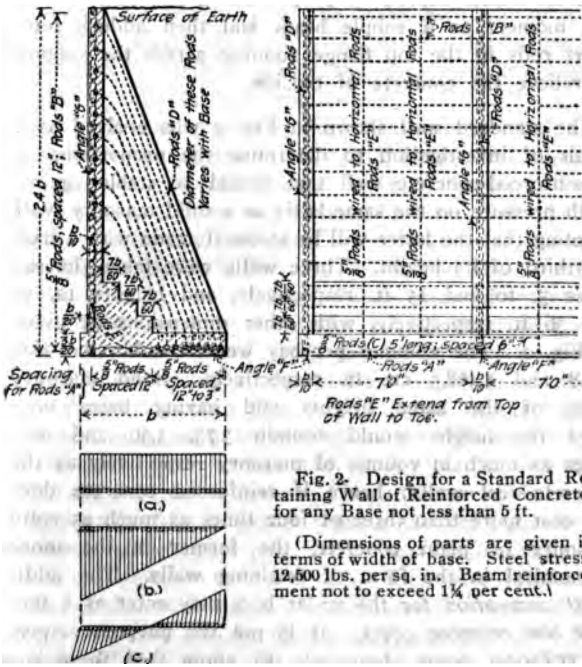


Fig. 2. Design for a Standard Retaining Wall of Reinforced Concrete, for any Base not less than 5 ft.

(Dimensions of parts are given in terms of width of base. Steel stress, 12,500 lbs. per sq. in. Beam reinforcement not to exceed 1¼ per cent.)

more. The reason for the 5-ft. lower limit of base and for the constant spacing of ribs will be shown in a later paragraph.

In designs that have been illustrated in engineering publications, two sets of rods are shown in the bottom slab, one near the bottom of slab and the other near the top, both spaced uniformly for the full width. The writer believes this to be unnecessary and uneconomical. The execution of the work is greatly complicated thereby, and the forces are not present demanding this reinforcement. While it is true that this slab is continuous, and while somewhat less moment is found by considering it as continuous, the purposes of good construction are better served by proportioning the slab for the full moment as a simple beam and then adding some short rods in the top flange running across the support to relieve the concrete of tension.

The standard wall shown in Fig. 2 was evolved as a result of investigation to determine the proportions of a reinforced-concrete wall that would be stable against earth pressure on the same basis as a solid masonry wall, granting that the latter will be stable if made with a base one-third of its height. Three walls were taken, having bases 5, 10 and 15 ft. respectively, and heights 12, 24 and 36 ft. respectively, with other dimensions as given in Fig. 2. The volumes per bay were found to be 96.6, 452.8 and 1,168.5 cu. ft. respectively. Solid masonry walls of the same heights and having bases one-third the height would contain 3.73, 3.39 and 3.14 times as much in volume of masonry respectively as the concrete-steel walls; hence if reinforced concrete does not cost more than three of four times as much as solid masonry or plain concrete, the former is the more economical in the form of retaining walls. The additional excavation for the wider base may enter as a factor in the relative costs. It is not the purpose to give actual relative costs, but only to show that there is a

large margin in favor of the reinforced-concrete retaining wall.

The centers of gravity of these three walls were found to be 1.43 ft., 3.00 ft. and 4.66 ft. respectively from the heel of wall. The respective volumes of earth over the slab are 332.3, 1,377.2 and 3,206.3 cu. ft. The centers of gravity of these volumes are located 2.96 ft., 5.97 ft. and 9.01 ft. respectively from the heel of wall. The moments of stability of these three walls, taken about a point one-third of the base from the heel, were found to be 39,599, 340,405 and 1,227,272 ft.-lbs. respectively, at 100 lbs. and 150 lbs. per cubic foot for earth and concrete respectively. The earth load considered is that directly over the bottom slab up to the level of the top of wall. Assuming that the force of the earth back of the wall is that of a liquid one-sixth of its density the weight of a cubic foot of earth to balance the above moments would be 110, 111 and 111 lbs. respectively.

One peculiarity respecting these three walls is that the resultant center of gravity of earth and concrete is in each case almost exactly in the center of the slab. This makes it proper to assume that the pressure due to the weight is uniformly distributed over the base, as shown at *a*, Fig. 2, as the stiffness of the bottom slab is sufficient to give this distribution. The reaction of the earth under the slab will be uniformly varying from zero intensity at the toe of wall to double the intensity of the uniform load at the heel of the wall, as at b The difference between these sets of forces, or the forces as shown at c, must be resisted by internal stresses in the concrete. On the left half of the bottom slab the forces are seen to be upward. If the construction were a reinforced slab, it would require the principal reinforcement in the upper part. For simplicity of construction it is desirable to avoid this. The close proximity of the vertical *wall makes* it possible to throw this force *directly into that* wall by means of the steps at the junc*tion of wall and slab.* These steps could of course be

replaced by a chamfer or slope, if the latter were found to be simpler of construction.

On the right half of the bottom slab there is a vertical load downward varying in intensity from zero at middle to an amount at the edge of slab which may be taken as the weight of superimposed earth and of the slab itself. This is close to $250 \times b$ lbs. per sq. ft. Taking the effective span of the slab as 7½ ft. the bending moment on the extreme right edge is $250\ b \times 7\frac{1}{2} \times 7\frac{1}{2} \div 8 = 1{,}758\ b$. This is in foot-pounds per foot width of slab.

In order to give clearance for the angles shown in Fig. 2, the steel rods are placed one-sixth of the depth of slab from the bottom. Following the method employed by the writer, and described in Engineering News, March 15, 1906, the effective depth of the slab is found to be 2/3 of its depth. But as the depth of slab is 3/20 b, its effective depth (that is, the distance from center of steel to center of compression in concrete) is 1/10 b. For an area of steel = A, in sq. in. per foot width of slab, and a stress of 12,500 lbs. per sq. in. we have an allowed bending moment of $12{,}500\ A \times 1/10\ b = 1{,}250\ A\ b$ foot-pounds.

Equating this to 1,758 b, found above, we have

$$A = 1.406.$$

This can be made up of four ⅝-in. diameter rods. It is thus seen that for any size of retaining wall ⅝-in. round rods spaced 3 ins. apart will take the tensile stress in the bottom slab at the extreme right edge.

When the area of steel does not exceed 1¼ per cent of that of the concrete in the slab (see article above referred to) the stress on concrete will be within safe limits. It is seen that the area A found above is 1¼ per cent of that of a slab 12 ins. wide and 9.4 ins. deep. For a base of 5 ft. the depth of slab is 9 ins.; hence a lower limit of about 5 ft. should be adhered to for this standard. For smaller walls the thickness of parts for a wall of 5-ft. base could maintained and the span between ribs varied to suit bending moment found. Thus, for a wall having t. base (one-half of the standard 5-ft. wall), the load

on the slab will be approximately one-half of that on the 5-ft. wall. (The height of this wall would be 6 ft.) A 9-in. slab with the standard reinforcement will be good for a span $\sqrt{2}$ times the standard, or 10.6 ft., say 10 ft. clear.

The span of 7 ft. in the standard wall is found as follows: Let s = clear span in feet. The end shear on one foot of width of the slab is $250\ b \times \frac{1}{2}\ s = 125\ b\ s$. The shearing strength of a slab 3-20 b feet deep and one foot wide, at 40 lbs. per sq. in. on the gross area is

$$3\text{-}20\ b \times 12 \times 12 \times 40 = 864\ b.$$

Then from the equation

$$125\ b\ s = 864\ b,$$

we find $s = 6.91$ ft.

Since this remains true for all sizes of standard walls, it is seen that the shear is taken care of also in walls of less than 5 ft. in base, as the slabs will have a greater relative depth than in the standard walls.

The reinforcement in front wall is found as follows: It will be seen in the standard retaining wall that the depth from top of wall to top of steps is $1.9b$. The intensity of horizontal pressure is therefore $1.9\ b \times 1\text{-}6 \times 100$ lbs. per sq. ft. This gives a moment on a foot depth of wall $= 223\ b$ ft.-lbs. The moment of resistance of the concrete-steel slab, found as above, is

$$12{,}500\ A \times \frac{2}{3} \times \frac{b}{10} = 833\ A\ b.$$

From

$$223\ b = 833\ A\ b,$$

we have $A = .267$ sq. in.

This would be made up by one ⅝-in. round rod for every foot of depth of wall. This reinforcement is carried up to the top of the wall and is used as indicated on the lower portion of the wall and on the bottom, though theoretically less reinforcement would be required. The purpose is to tie the wall together.

The rods in the rib act to tie together the bottom slab

and the vertical wall, the manifest tendency being for these to pull against each other. The chief use of the concrete in the ribs is to protect these rods. It is essential that the rods have an efficient end anchorage, and any other anchorage or bond can serve only a subsidiary purpose. The line of cleavage in the rib may be immediately above the bottom slab, and practically the whole strength of the rod is needed at that plane. In the short depth of the slab there is not length enough for even mechanical bond to take effective hold for the full strength of the rods. Hooks or curves on the ends of rods have not been shown by tests to be effective end anchorages. Such a detail would not be accepted in structural steel work. The use of plain round rods with thread and nut at end, attached to a front-to-back angle in the bottom of the slab, as shown in Fig. 2, seems to be the most economical as well as the most effective means of meeting these conditions. These angles may also act as anchors for the horizontal rods, at ends of wall, if any such anchorage is needed.

The angles serve to locate the rods properly and to hold them in position against displacement during the placing of the concrete. They also make it more difficult to omit any rods and easy to detect any omissions. All of these points are of great weight in construction where unskilled labor is so commonly employed.

The rods in the rib take the downward force shown at c, Fig. 2, the amount of which at the right edge is at a rate per foot close to $250\ b \times 7 = 1{,}750\ b$ lbs. On a retaining wall of 5-ft. base the load in a foot is 8,750 lbs. At 12,500 lbs. per sq. in. this would require four rods ½-in. in diameter. For a 10-ft. base four 11-16-in. round rods are required, etc. By giving these rods the same horizontal spacing at bottom as those in the bottom slab and varying the rate of spacing in the same manner, the *vertical load* is taken care of.

These rods start vertical; hence the vertical load

measures their stress and not the component in a diagonal direction. Their stress is not uniform throughout, but they begin near the foot of the wall to transmit their stress into the concrete to resist the upward thrust of the forces on the left half of the base. Their radius of curvature should not be less than about twenty times their diameter, so that the unit pressure exerted by the side of rod on the concrete will not exceed safe limits. (A square rod curved to a radius twenty times its diameter will exert a radial force on its side one-twentieth of the intensity of its tension, which is the usual ratio between the allowable stresses in steel and concrete. The use of sharp bends in embedded rods is a structural fault often met with.)

The rods should have two nuts on the ends, so that the end nut can be drawn to a tight bearing on the angle. Horizontal rods should be spliced by sleeve nuts.

A modification of the design can be made in which the rods in rib pass through slotted (or large sized punched) holes, and have cast iron beveled washers. The rods can then be straight. The inclination would be such as to give about 8 per cent more stress than found in vertical rods.

The gripping of concrete is sufficient to make up for the difference in strength of rod at root of thread and that of the full section.

Anchors that take the full stress of rods should have their areas twenty times the section of the rod. This condition would govern the size of the flange of angle to which rods in the rib connect. These angles could vary from about $2\frac{1}{2} \times 2\frac{1}{2} \times$ 5-16-in. for small sized walls to $4 \times 3 \times \frac{3}{8}$-in. for large walls.

The formulas ordinarily given for continuous beams are for beams having a uniform moment of inertia. If the beams have not a uniform moment of inertia, the formula does not hold. If a beam be purposely designed so as not to have a *uniform* moment of inertia, it will deflect *more at the section not* reinforced for the full moment,

and thus put greater moment on the fully reinforced section. This is the basis for the somewhat arbitrary reinforcement of the upper part of bottom slabs. The bending moment at the middle of this slab cannot exceed that for a simple span. The upper rods are added to prevent cracking, and consequent weakening of the slab in shear, near the ribs.

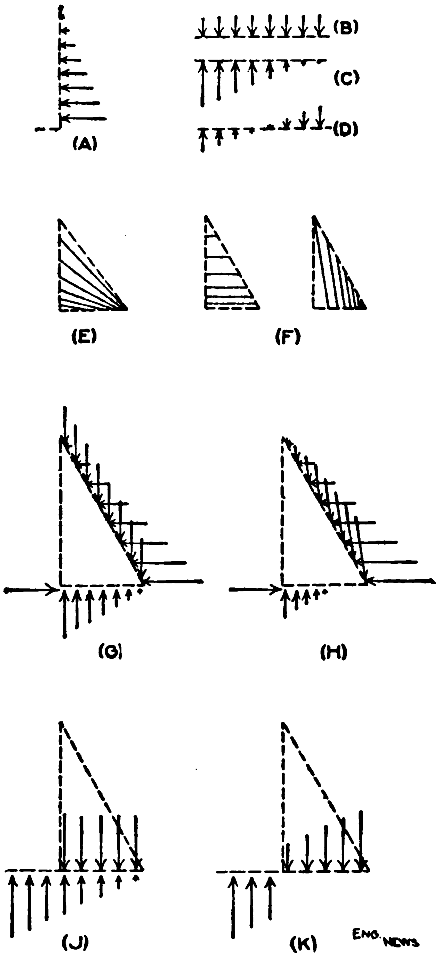
It is recommended that long walls be concreted from one end to the other, and not uniformly from the ground up; so that the shrinking of the concrete as it sets will not act on the whole length of the wall at once. It is also recommended that at expansion joints, say about every 100 to 150 ft., a complete separation be made in the wall, with an end rib in each portion of the wall.

THE DESIGN OF REINFORCED-CONCRETE RETAINING WALLS

Sir: The article in your issue of Oct. 18, by Edward Godfrey, on the "Design of Reinforced Concrete Retaining Walls," was of especial interest to the writer, because it so closely conformed to the methods of design lately developed by him in checking the computations for a very high and long wall about which he was consulted.

Differences of opinion may exist as to proper methods of determining the lateral pressures exerted by earths of different qualities, and as to the proper design of reinforced concrete beams, etc., but certain fundamental principles exist in accordance with which every proper design should be prepared, and it seems to the writer that Mr. Godfrey has come nearest to stating them of any one with whom the writer is acquainted.

The writer's experiments on lateral earth pressures, reported to the American Society of Civil Engineers, led him to the opinion that the angle of surface repose for most earths is entirely different from the angle of internal friction of the same earths. Manifestly it is the latter


(A)
(B)
(C)
(D)
(E)
(F)
(G)
(H)
(J)
(K)
ENG. NEWS

which should be used in most design work instead of the former. The writer agrees with Mr. Godfrey, however, that the actual active pressures may with sufficient accuracy be considered as fluid in effect, and a certain percentage of the weight of the earth backing in amount. The experiments above referred to seemed further to show that for walls more than about 5 ft. high the pressures varied practically as the depth, so that their resultant might, with sufficient accuracy, be considered as acting at the lower third point of the height of the wall. For walls of less height, however, this point of application rose often to a point four-tenths of the height above the base. In most walls, then, the lateral pressures would be represented by a triangular diagram of horizontal arrows, having a horizontal base (sketch A).

The writer is in direct accord with Mr. Godfrey where he uses a bottom slab weighted by the superimposed earth, and agrees that the resultant downward pressure of the earth on this slab may, for all practical purposes, be considered as uniform (sketch B) unless the design employs a greatly extended toe, as is often necessary with high walls on soft soils.

He also agrees with Mr. Godfrey that many designs are faulty in the arrangement of the reinforcement in the bottom slab, probably from wrong assumptions as to the distribution of the resisting earth pressures.

While some experiments lately carried out by the writer (which are now being repeated) shed some new light on the distribution of resisting earth pressures under walls and foundations, still for the moment a uniformly varying distribution may be assumed, and, provided the resultant of the two series of forces above described intersects the base of the wall at the front edge of its middle third, the variation is from zero at the back to twice the average at the front (sketch C).

The diagram of resultant forces on the slab will then *evidently be* two triangles, as described by Mr. Godfrey *(sketch D).*

The horizontal forces are, of course, resisted by the sliding friction between the base of the wall and the earth beneath, and by the pressure of the toe of the wall against the earth in front of it. This front pressure may be considered as distributed uniformly over the base like the friction, for sake of convenience.

These assumptions take care of all the external forces. If the slabs in front and base are to be tied directly together by diagonal rods in the counterforts, these rods must be given varying slopes (approximately as at *E* in the sketch herewith) to correspond with the varying pressures in front, while their bottom ends must be similarly spaced to correspond with the resultant downward earth pressure. Then, too, these rods would have quite different stresses due to their varying slopes, so that the proper distribution of diagonal rods of uniform diameter is a very complicated affair to determine. Evidently, if some satisfactory arrangement of horizontal and vertical rods can be devised to produce the same effect several advantages will be gained. Even the arrangement proposed by Mr. Godfrey (the correctness of the design of which the writer strongly questions) is rather costly in execution, because simple horizontal and straight, approximately vertical rods are much more easily placed by inexperienced labor.

Such horizontal and vertical rods in the counterforts, if spaced to correspond with the assumed forces, will have a uniformly varying spacing one way to correspond with that which theoretically should exist on the face, and a similar one in an approximately vertical position the other way, the upper ends of the rods running to all points along the back of the counterfort, while their lower ends cover that part of the base over which the resultant earth pressure is assumed as acting downward (see sketch F). These rods simply carry to points within *and along the backs* of the counterforts the active earth *stresses, so that the* resultant diagram may be drawn

as shown at G when primary stresses alone are considered, and as at H when the resultant earth pressures and rod actions are considered.

Now the concrete in the counterforts being highly resistant to compressive stresses, easily resists the combined action of these loads and the whole counterfort is much better tied together than in Mr. Godfrey's design. The latter seems a little inconsistent in not better reinforcing the counterforts after employing more steel than theory requires at several other points. A similar inconsistency crops out in the omission of any steel in the steps which are supposed to carry over into the face of the wall the upward pressure near the front under the bottom slab. Unless the concrete is considered as taking some tensile stress, reinforcement should be employed in some manner in these steps, either to make them act as beams of variable depth carrying the upward pressure to the counterforts, or as brackets attached to the inside of the front face. Under the latter assumption horizontal rods perpendicular to the face should be introduced in the bottom slab. These rods should also be carried through the plane of the face and into the toe projection, which must often be greatly extended, and the design of which Mr. Godfrey does not consider even though he shows such a toe in his diagrams.

If the remainder of the bottom slab outside the stepped portion, besides acting as a vertical beam to take earth pressures, is considered as a horizontal beam designed to resist the stresses which the lateral rods might be introduced to resist; then proper additional ties should be introduced in the counterforts. Mr. Godfrey's angle iron would not seem of sufficient size to cover the usual needs.

It is believed, further, that the practical construction of the forms for Mr. Godfrey's stepped wall will be more costly than for a wall without the steps and with steel introduced instead.

Whenever the weight of a cubic foot of the earth backing multiplied by the height of wall exceeds half the safe

bearing power of the soil, an extended toe must be employed. In that case the diagram of vertical stresses may be assumed as in sketch *J* and the resultant downward effect on the bottom slab may or may not reach zero behind the face. A diagram somewhat like that of sketch *K* will result.

In this case the rods in the counterforts which are designed to carry these vertical resultant stresses will themselves be nearly or quite vertical. The exactly vertical condition would occur when the resultant earth pressure becomes zero under the face of the wall, which condition would take place when the toe is extended a distance outside this point approximately three-tenths of the total width of base. This amount may seem rather large, but is not unknown.

With such an extended toe a series of steps in the outside of the wall would be very proper because no counterforts are available, and since the face of the wall is in close proximity. Properly arranged steel would be very necessary unless tension is to be allowed on concrete.

What Mr. Godfrey says wth regard to end anchorage of rods, the writer believes to be eminently correct, except that the writer's experiments have shown that proper hooks are entirely satisfactory. The writer further agrees with Mr. Godfrey in always advocating steel in beams and slabs to take up reverse moments at points of support. except that he feels that the design of the slabs as simple beams with extra reverse reinforcement added, is erring too much on the side of safety even with regard to the exceedingly uncertain knowledge we now possess as to earth pressures and reinforced concrete continuous beams.

The writer has been greatly interested in Mr. Godfrey's design, but considers that the one briefly described above is more logical and more economical of execution, especially as to labor. Yours truly, E. P. Goodrich.

1170 Broadway, New York, N. Y., Oct. 18, 1906.

THE DESIGN OF REINFORCED CONCRETE RETAINING WALLS

Sir: Kindly permit me space to reply briefly to Mr. E. P. Goodrich in his criticism (Eng. News, Nov. 15) of my article on design of reinforced concrete retaining walls (Eng. News, Oct. 18, 1906).

As to the placing of straight horizontal and vertical rods being simpler than the inclined rods used by me, my idea would be to have the rods and angles sent cut and punched and threaded to the site. The harp-like arrangement in each rib or counterfort can be set up on the ground and raised to position. A few braces would hold all in place, whereas, with all rods separate, each would have to be held individually, and liability to displacement is multiplied.

The total pressure against the front slab is less than that against the bottom, hence with the same rods, starting normal, there is more than enough strength to take all of the force. The occurrence of rods apparently closer than necessary near the top of front slab will give an extra safeguard against such pressure as that due to freezing of the ground, which would be greatest near the surface. Extraneous load would probably effect a lateral pressure near the surface only.

In the matter of the projection in front of the wall, and of the steps being without reinforcement, it is seen by my sketch that the front projection is only one-third as broad as its height; also the uplift under the steps tapers down to nothing at the foot of the steps. There is strength enough in the concrete to take the resultant tension at a very low value. Spread footings are allowable even in brick or rubble masonry; they ought to be so in monolithic concrete. If there were necessity for a large projection in front of the wall, a modified design would be required. It was my purpose to give a rational analysis of the stresses and rational methods of taking care of them. Yours very truly, Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., Nov. 19, 1906.

A Method of Measuring Deflections in Floor Tests.

[Published in Engineering News, Aug. 25, 1904.]

By the Author

The following description of the method employed to measure the deflection of a floor under test may be of interest to any who have similar tests to make.

The floor tested was in 20-ft. squares, and it was desired to obtain the deflection at the middle of the square tested, as well as at the middle of the side of the square, or half way between the columns. To accomplish this, pedestals or uprights were made of a single 4 x 4-in. piece of wood nailed to a block about 2 x 12 ins. by 2 or 3 ft. long. These were placed on the floor at points where the deflections were to be measured, and blocks of pig iron were placed upon them to weight them down, so as to prevent displacement by the men in loading the floor. Enough pig iron was placed on each pedestal to make up approximately the required load for the space which it occupied.

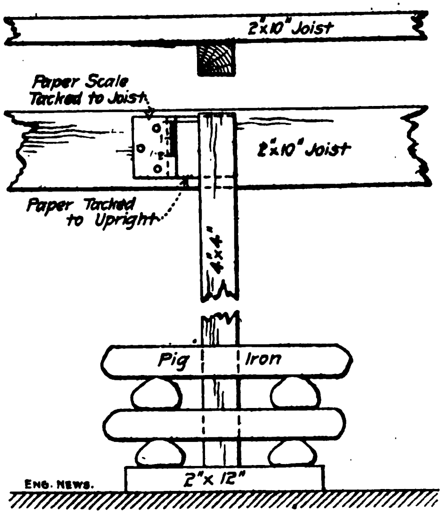
In addition to the pedestal there had been placed 2 x 10-in. timbers which reached across the floor space and were nailed to posts resting on the floor near the columns. These were about at the level of a man's head, so as not to be in the way of the men who were loading the floor.

They were vertically over the points at which deflections were to be taken. The uprights were placed so that a flat side of the 4 x 4 was against the joist or timber which reached across the floor. On the back of the upright there was tacked a sheet of stiff paper, upon which was ruled a horizontal line. On the joist was tacked another sheet of stiff paper, *the edge of* which had been divided accurately *into tenths of an inch.* This was set with the zero of the

scale at the horizontal line on the sheet tacked to the upright.

As the floor deflected, the amounts of the deflections could be instantly read on the scale to hundredths of an inch, by estimating tenths of scale divisions. The apparatus is more clearly shown in the accompanying sketch.

A number of scales were ruled at the same time by tacking the sheets down on a drawing board with about ⅛-in.



Apparatus for Measuring Deflections of Floor Panels in Load Tests.

along the edge of each exposed. By dividing one sheet with a decimal scale and ruling across the exposed edges all were made alike. The tenths could be further divided into five parts or fiftieths with a hard pencil, but no difficulty was experienced in estimating the hundredths with a scale divided into tenths.

Reinforced Concrete Engineering in the Making.

Concrete-steel or reinforced concrete construction is only an infant as yet. The relatives have not agreed upon a name. Many want the hyphenated appellation with the old family name of steel retained, while others would express the office of the steel rather than its presence.

To drop the figure, there is much to learn by the builders of this construction, some things that only experience will teach, and others that can be wrung from known facts and technical knowledge in kindred lines. It is the purpose of this article to point out some of the things that ought to be set up as guides in the design and execution of reinforced concrete constructon, if it is to have a standing in the class of sound engineering.

While it is true that the manufacture of reinforced concrete can be accomplished largely with ordinary labor, it is also true that this labor must have strict supervision by competent foremen, who understand the importance of doing the work just as the designer has planned it. A laborer does not understand the importance of a small rod in the concrete, and would probably see no harm in leaving some rods out; or he might think that the exact location of rods is a matter of no importance, so long as they are present. The displacing of rods, either by accident or design (as to make the placing of concrete more convenient), may be the result of an ignorant workman's act for which he would feel no guilt because of his ignorance. Rods that are intended to lie close to the bottom of a beam or slab may thus be placed at the middle of its depth, resulting in a great reduction in the strength, and not improbably being a cause of failure. Rods bunched together, where they should be separated, is a possibility that would result in a loss of gripping power in the concrete.

Again a laborer may wish to save himself the handling of cement and cut down on the amount used; or, with a view of saving his employer expense, with the latter's con-

sent, a bag or two of cement may be left out occasionally. The time of mixing or number of turnings on the mixing board, if hand mixed, should not be left to any but experienced and responsible persons. The uniformity of the concrete depends upon the thoroughness of mixing and correctness and constancy of the amounts of the ingredients. The strength of the structure is gaged, in a large measure, by the uniformity of the concrete.

An illustration of a workman's idea of the possibilities of concrete is found in the following: The writer observed some work being put in, where three or four inches of plain concrete was being laid on an old wooden floor. With the idea that the floor had probably been shored or strengthened for the heavy load he asked the workman what supported the concrete. The reply was, that "the stuff didn't need any support, it supported itself."

Inspection of every part of the work as it progresses, by someone not interested in the contractor's end of it, is almost an absolute necessity The combination of possibilities of skimping and scamping, through ignorance or carelessness, leaves the owner with the small end in the probability line, unless he adopts suitable means to correct this condition. A contractor's guaranty that a structure will be built according to specifications, or that it will not crack or deteriorate in a given period, is scant comfort when a piece of work shows defects upon its completion, and the contractor's final estimate declares that the structure is complete and ready for use.

A guaranty that a structure will stand a specified test load is another uncertainty to which owners sometimes tie. A test load on a small square of a large floor of continuous construction is no more a criterion of its capacity when fully loaded, than the ability of a tank to hold a barrel is proof that it will hold ten barrels. In the early days of bridge building it was not unusual to see in specifications a requirement that a bridge should be loaded for a certain period with a given train load. Bridge builders have advanced away beyond that point. It is the man in the de-

gning department that can tell to a nicety what the bridge fit to carry, the presumption being that all parts of the brication of the bridge have received the necessary in-pection, and the plans are carried out in every particular.

Tests on reinforced concrete construction are quite roper, but they should be made on an isolated unit of the oor not supported on all sides by the contiguous con-truction; or else a test to be of value, should be made on large section of the floor in place, of sufficient extent not o be affected by extraneous support.

One contract which came under the writer's notice called or floors that had a theoretical breaking strength of three imes a certain load per square foot. The floors in ques-ion proved to be unsafe under test with one time this load per square foot, though by the designer's method of figur-ing the "theoretical breaking strength" was supposed to be three times. The theoretical breaking strength of concrete or of a concrete-steel floor is too nebulous to have any meaning in a contract.

An error that works to the detriment of reinforced concrete construction is the notion that it is the cheapest form of construction. For many reasons it is economical, but in no sense is it cheap. Properly built it cannot compete, even at present high prices, with wooden construction in the matter of first cost. It compares favorably in cost with steel construction, and in many situations structures can be built of reinforced concrete for less than of structural steel, when the same character of design is maintained.

The necessity for using wet concrete cannot be too strongly urged. Dry concrete is lacking in the essential characteristics that make the combination of steel and concrete so strong and durable. Mealy concrete will be porous and fail to protect the steel; wet concrete, if it contains enough cement, will coat the steel with a film of cement. This is one essential to the preservation of the steel; dense concrete is another. Neither of these are possible with dry concrete. Dry concrete has not much adhesion; it will therefore fail to take hold of the steel. It is also lacking

in cohesion, and would therefore be weak in its gripping power on the steel. Dry concrete will set in a shorter time than wet concrete, and on short time tests will show greater compressive strength. The wet concrete will, however, attain greater strength than the dry concrete when it has thoroughly set. Many erroneous notions about the strength of reinforced concrete and the preservation of the imbedded steel have no doubt been the result of the use of dry concrete, and the unsatisfactory conditions observed. Concrete for this class of construction should be puddled rather than tamped, by means that will work out the air spaces and make the concrete to run into all crevices.

In the matter of materials and the proportions of the same there is not much difference of opinion. Good, hard and durable broken stone or gravel is essential in stone concrete for any purpose. For reinforced concrete the stones should be small, say under an inch in every dimension for slabs, beams, columns, and small arches, and larger in size for more massive work. The reason that small stones should be used is that they will pack better around the steel and not leave voids. Graded sizes from the largest to the smallest are very essential both in the stone and in the sand. Uniformity in the sizes of stone or grains of sand is to be guarded against. The larger parts leave voids that only the smaller parts can fill, and these leave voids that still smaller parts are necessary to fill, and so on down to the finest particles of cement.

In plain concrete economy can be effected by a mixture that is suited to the particular broken stone or gravel used. In such concrete it is only necessary to provide enough sand to fill the voids in the stone and enough cement to fill the voids in the sand. In reinforced concrete there must be an excess of cement, so that the density of the concrete will be assured, and so that the steel will be covered with cement.

One part of Portland cement to two parts of sand and *four parts* of broken stone, gravel or cinders, all by volume, *is the* generally accepted standard mixture. A leaner

mixture than this is not recommended. Stone concrete of these proportions, made of good materials, will have a compressive strength in short blocks of about 2,000 pounds per square inch. To have a factor of safety of four, as it should have, a compressive unit stress or an extreme fibre stress in beams of about 500 pounds per square inch should be employed in the design. Concrete in short blocks can be stressed to this amount with safety. It is therefore a proper unit for reinforced concrete columns or beams. It would not be safe in a plain concrete column, because a plain concrete column (having a length several times its diameter), will fail in other ways than in direct compression, such as bulging or diagonal shear. But where the concrete is tied together by steel in such a way that the concrete is not subject to any but compressive strains in short braced elements, failures by bulging or by diagonal shear are prevented.

Steel buildings are commonly built in lengths of several hundred feet without expansion joints, depending upon the elasticity of the steel to take up temperature stresses. Reinforced concrete builders, essaying to do the same thing with the less elastic materials, are apt to have serious trouble. If a long wall or building is brought up from one end to the other, so that the shrinkage of the concrete does not act on the whole line at once, it is believed that one or two hundred feet of continuous reinforced concrete can be maintained in one piece. The placing of the concrete should be done from one end to the other of a long structure where possible. It is well to have expansion or cleavage joints and two sets of columns in very long buildings. So-called expansion joints, where the two parts of the building coming together are not independently self sustaining are worse than no provision whatever for expansion. Two sets of girders on one column with an expansion joint between them would mean the concentration of all of the temperature stresses on the columns.

It is very important that, excepting at expansion joints, *structures be tied together* continuously. A break in th

continuity of the steel results in a weak point or section in the structure under temperature stresses.

In the matter of the time allowed for the setting of concrete the practice common with plain concrete will not apply in reinforced concrete. In plain concrete the forms can often be removed in a day or two after the concrete has been placed, and no harm results. This is because the stresses in plain concrete are not of the intensity of those in reinforced concrete, and because of the further fact that plain concrete seldom receives its calculated load until the structure it supports has been built up over it, which may be months later. The dead weight of reinforced concrete beams and slabs is a large part of the total load which they carry, hence they should not be called upon to support their own weight until the concrete has attained the greater part of its calculated strength. In a current engineering periodical, description is given of a building in which the statement is made that forms were removed from some very large girders two days after the concrete was placed. There is absolutely no warrant for such practice. The straining of the steel rods before the concrete has firmly gripped them is a proceeding that is fraught with great danger to the safety of the structure. No wise builder would submit to his structure being subjected to a test two days after the concrete had been placed. It is just as absurd to remove the forms at so early a date. Two weeks of good weather should be allowed before forms are removed, and two weeks more should elapse before any test is made. In freezing weather a longer period should be allowed, as the setting of cement is retarded and sometimes almost suspended in freezing temperatures.

If construction is proceeding upwards, as in buildings, speed should not be too great. If too much weight is placed upon the columns while the concrete is green, damage will result. Forms for concrete columns should be well tied together with a view of containing the semi-liquid concrete as well as resisting the bursting pressure due to the load that may come upon the column. Props should be

placed close to columns as well as under intermediate points in the beams and girders. Column forms, as usually constructed, are not suited to taking vertical loads, but are built merely as boxes to contain the concrete. The centering or props should be strong enough not only to support the concrete in the forms immediately above, without sagging, but also to sustain whatever load may come upon the same during construction for the time that it takes the concrete to set.

The qualities that make the combination of concrete and steel not only a possibility, but also a means of meeting engineering problems that in many ways has no equal, are these: Concrete and steel have nearly the same coefficient of expansion under changes of temperature. Concrete in setting in the air will shrink and grip the steel; it will also adhere firmly to clean or somewhat rusted steel, (but not to oily steel). Steel imbedded in concrete will supply the tensile strength that the concrete lacks, and the concrete will preserve the steel against rust besides acting as a protection against fire.

That the coefficient of expansion of the two materials is not quite the same is reason for predominance of the weaker material. Large sections of steel in comparatively small sections of concrete should not be used, as the expansion and contraction of the steel will crack the concrete. The need of mass in the concrete to grip the steel to the capacity of its tensile strength is another reason why the steel should occupy only a small fraction of the sectional area of a beam or slab.

The placing of steel directly against the forms and thus allowing it to be on the surface of the beam or slab is bad practice. The steel is by this means exposed to fire and rust, and it cannot be effectually gripped. Steel rods should be several times their diameter from the surface of the concrete. Beams narrow at the bottom and wide at the top are irrational in shape. They lack in concrete just where it is needed to protect and grip the steel. So-called T beams, which are rectangular beams, including in their

calculation a portion of the floor slab as top flange, are faulty in like manner, in that they do not contain enough concrete in the lower part of the beam for the amount of steel used.

Floor plates in steel construction are not considered as adding to the strength of floor beams, even though firmly riveted thereto. It is not good engineering to consider a wide expanse of floor slab as part of a narrow beam, and it is not doing justice to the owner to make a floor slab which he cannot cut into, for the many necessary purposes that openings are often made in floors, without weakening the primary support of his floor.

Wide flat bars bedded in concrete are not held as firmly as square and round bars of the same sectional area, and are not suitable shapes, for the reason that the concrete tends to shrink away from the flat sides. Flat bars have the further disadvantage that they make a plane of cleavage in the concrete. This is especially true if they are near the surface. The writer observed some reinforced concrete beams in which several flat bars were placed side by side and brought up and hooked over the steel beams. When the forms were removed, large chunks of concrete below the flats fell off. Square and round rods should therefore be used in preference to flat bars.

The writer has no quarrel with mechanical bond, but he believes that more is to be gained by use of plain commercial steel and a low unit tension than by using mechanical bond and the high unit tension advocated by those who have the special material to sell. If the question of economy cuts no figure, the use of deformed rods under low unit stress will add an element of safety, which is more in the nature of insurance against bad work in the execution than a needful precaution in the design.

The proper use of the steel is to take tensile stresses. Composite structures, such as the combination of wood and iron in a beam, are poor makeshifts at the best. Where dissimilar materials are used to perform the same office jointly there is no correct way of determining just how

much of the work each will do. A column having longitudinal sections of steel that are intended to share in supporting the load is a composite of concrete and steel and not a true reinforced concrete column. Calculations to determine the relative amounts taken by the steel and concrete are rendered useless by reason of the tendency of the concrete to shrink and shorten.

Segmental floor arches, that is, arches flat on top and curved upward on the under surface, with steel reinforcement near this curved surface, violate the principles of good design. If these act as arches and not slabs, they need tie rods where they would pierce the curved surface and be unsightly in a ceiling; the steel is in compression in place of tension. If they act as slabs, they are shallow and weakest where they should have the greatest strength.

The steel in reinforced concrete should be in comparatively small sections, well distributed through the mass. Not only should there be the steel which calculations show to be required for tension, but it is very often desirable to use steel rods at right angles to the primary rods in order to tie the concrete together and prevent cracking. These cross rods also aid greatly in the lateral distribution or spreading of concentrated loads, as in arches or slabs. Wire mesh, in which the wires are straight, is an excellent material for floor slabs of small span; as uniform spacing of the steel wires is assured, and they are not easily displaced in placing the concrete.

If wires or rods are bent or kinked, the tendency of stress in the same will be to straighten out the bends and kinks. This means excessive stretch in the steel accompanied by cracks in the concrete. The use of wire cables is irrational. It is a well known fact that a wire cable will stretch several times as much as a steel rod of the same sectional area under the same load. The principal reason for using wire cable must be because of the high tensile strength of the steel. Now if a high unit is used in the cable, the stretch will be still further augmented.

Rods imbedded in concrete should not be given angular

bends. If a rod under stress is bent at an angle, there will be a large component of stress at the bend in a direction bisecting the angle between the portions of the rod. There is evidently nothing at the bend to take this component but a small area of concrete bearing on the rod at the bend. The use of sharp bends in rods is a very common fault in reinforced concrete design. It is inexcusable. These rods should be given gentle curves, so that the side bearing will not be excessive. If we allow a side pressure on the concrete one-twentieth of the tensile unit on the steel, we may arrive at a safe minimum radius of curvature of a rod as follows: Let d = diameter of a square rod, and p = unit pressure allowed on the concrete. Then 20p will be the unit tension on the steel, and $20\ pd^2 =$ stress in rod. But the stress in the rod must equal the product of the width d of the rod, the radius of curvature r, and the unit pressure on the concrete; or $20\ pd^2 = d\ r\ p$, whence $r = 20d$. That is, the radius of curvature of a square rod should not be less than 20 times the diameter of the rod. The same rule may be applied to the case of round rods without important error.

Hooks at the ends of rods as a means of end anchorage have the same structural fault as sharp bends. They cannot anchor a rod for its full tensile strength. A proper anchor for a rod requires a washer plate or other bearing part having a surface in bearing against the concrete about twenty times the area of the rod.

No tension on the concrete should be allowed in the calculation for the strength of beams or slabs. The tensile strength of concrete is uncertain and unreliable, especially where it may be subject to expansion cracks due to the presence of the steel. One crack in a beam may destroy entirely an assumed tensile value, although the same crack might not be a serious matter in a beam not calculated to receive any assistance from the tensile strength of the concrete.

The allowed tensile unit on the steel is a feature of design that has not received the attention that it merits. The

mere integrity of the steel under a given tension per square inch may be taken care of, and yet the stretching out of the steel under that stress may be such as to disintegrate the concrete. Cracks begin to appear in the concrete after the stress in the steel has passed about ten or twelve thousand pounds per square inch. Units of about these amounts should therefore be used. The use of high tension on the steel is indefensible. There seems to be a prevailing opinion and it is held by many men who ought to know better, that high carbon steel or steel that by cold rolling or otherwise is made to have a high elastic limit will elongate less under stress than ordinary structural or soft steel. This is entirely erroneous. For stresses below the elastic limit all grades of steel have practically the same modulus of elasticity, that is, they will all stretch out a given amount under a given unit stress. Beyond the elastic limit the soft steels will elongate more before breaking, but the properties of steel under stresses beyond the elastic limit have nothing at all to do with the design of reinforced concrete. The writer has in mind a case where an architect wished to obtain rods having an elastic limit of 60,000 pounds per square inch, so that he could use 20,000 pounds in his design; because the building laws allowed him to use a factor of safety of three on the basis of the elastic limit. In another case a designer used 32,000 pounds per square inch for dead load stress on the steel. (This information was given in the defense of the design of a bridge that failed.) There is absolutely no warrant for the use of such high stress upon steel confined in concrete. One advocate of the use of high steel, and mechanical bond, makes the defense that the mechanical bond will distribute the cracks; thus virtually admitting that the high tension he advocates will result in cracks in the concrete. Public confidence is not obtained by any such admissions or practices.

Shear in steel, while it has a conspicuous place in specifications and building codes, has practically no meaning in *reinforced concrete. For* a steel rod to be in shear to the *extent of 12,000 pounds* per square inch, as sometimes

[illegible] the shearing strength of t[illegible]
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[illegible] of the concrete must
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tural designer; and to the extent that the personal equation is eliminated by these rules is the design perfected. Common sense and professional judgment sound well, but they are too often just another way of expressing the working of the process colloquially known as skinning. In light steel bridges it is not uncommon to see principles of good engineeering thrown to the wind. It will not do to follow the same course in structures of the permanence of those in reinforced concrete. A reinforced concrete structure with no better provision for rigidity than the average highway bridge (put up with no disinterested engineering supervision), will very soon shake itself to pieces. The highway bridge has the advantage in this respect because of the tougher material of which it is made.

The sooner reinforced concrete design is reduced to rule, and the more inclusive the rules, the better for the preservation of life and property.

In the matter of methods of calculating the strength of beams many formulas have been brought out, the great part of which are very complex. Complex formulas are not consistent with the nature of the materials nor with the result of tests. Complex formulas lead to errors, not only on account of the difficulty of applying them, but because of the blind way which they are generally applied.

By assuming the neutral axis always at the center of depth of the concrete beam and compression in the concrete as uniformly varying from the neutral axis up, the formula for bending moment, as well as its derivation, are rendered very simple. The first of these two assumptions is quite rational, because of the fact that tests to locate the neutral axis of beams under safe loads have shown that it lies very close to the middle of beam. The second assumption is equally sound. By the principle of the equality of tensile and the compressive stresses in a beam, if 500 and 10,000 pounds be used as extreme compressive stress in concrete and tension on steel respectively, it can be shown by a *simple* calculation that there will be 1¼ *per cent of steel area in the* beam. It is not the purpose in

this article to give formulas, but to lay down general principles.

The common form of beam has one or more rods near the bottom from end to end. These rods receive their stress by increments from the concrete. These increments come in the form of horizontal shear in the concrete, being transformed from horizontal shear to stress in the rods by the medium of the gripping power of the concrete. As the web of a plate girder must have sufficient shearing strength in any given length to take the flange increment in that length, so there must be section enough in the concrete beam to take the increment of the flange in a given length. The shear on the concrete in a horizontal section just above the rods must be provided for in section in the beam. This is one of the reasons why narrow beams and beams that are lacking in concrete in the lower part of the rectangle are faulty. If the gripping power of the concrete is measured by the area of the rod in contact with the concrete, and that gripping power is taxed to its safe capacity in any beam, the area of the horizontal section in the beam should be equal to the area of the surface of the rods. This means that square rods should be placed four diameters apart and round rods 3.1416 times their diameter apart.

As intimated, the holding power of concrete on a rod is measured by the area of rod in contact. This cannot exceed, in amount per square inch, the shearing strength of the concrete, for it is clear that a prism could shear out, taking just the skin of concrete adhering to the rod. The adhesion and shear are usually taken at about the same unit value. A safe amount is 50 pounds per square inch for stone concrete. It will be seen by a little calculation that, if a rod is imbedded fifty diameters in concrete, at this amount per square inch of its surface, it will be anchored to the extent of 10,000 lbs. per square inch. Hence, any plain rod not anchored with nut and washer at the end must be imbedded fifty diameters before it is held by the concrete to the amount of its safe capacity. In the case of a beam, however, while the anchoring value increases uniformly to

ıe center of the span the stress increases as the ordinates : a parabola. Hence, in order to have the anchoring value ɔt less than the stress at any section the rod should have ɔuble its safe anchoring strength at the center of span, nce the tangent to the parabola at the ends of beam cuts point twice as high as the middle ordinate at the center : span. This would require 100 diameters up to the cen- r of span. In other words a rod not anchored at the ıd of span, or continuous into the next span should not ιceed in diameter one two-hundredth of the span.

The curving of a rod up to the top flange at the ends : span, without providing an end anchorage or running ıe rod into the adjacent span for anchorage, is another ructural fault, and a common one. This takes the rod ıt of the plane where flange increments are added (near e bottom of the beam) and makes a suspension rod : it on which the concrete rests as a saddle. It is neces- ry in such a case that nearly the full stress of the rod ıould be imparted to it at the very end, where in many ·signs there is absolutely nothing to perform this office. a rod curved up at the end of one beam runs beyond e support into an adjacent beam, it serves the double ırpose of anchoring the rod and taking tension in the p flange of the next beam that would be caused by the ntinuity of the beams over the support. Separate rods ing near the top of beams, where two beams occur ıd to end over the same support, are of great value in ·sisting negative moments over supports, and should : made use of in the absence of the arrangement just entioned (where the bottom rods of one span continue to the next to resist tension in the top flange). These parate rods should extend not less than 50 diameters to each beam for anchorage, and may require to be nger. Rods running from one beam into the next r anchorage should extend into that beam not less an 50 diameters.

In steel work beams are not usually considered as ntinuous, unless it be necessary to take care of canti-

lever stresses, but in steel beams there is ample strength in the top flange to take care of any tension that may occur due to continuity. With reinforced concrete beams the case is quite different. Where two beams in a line resting on the same support are poured at once, there will be sure to be some tension in the top flange; and if this is not resisted by steel, a crack may be the result, which would destroy the shearing strength of the concrete, and might cause the beam to fail by shear.

The end shear of a reinforced concrete beam or slab with only horizontal rods must be taken by the concrete. When a beam of a given span is increased in depth, the bending strength and consequently the capacity for load is increased about as the square of the depth. The shearing strength is, however, only increased as the depth. There will therefore be a point where the shearing value is overtaxed, and beyond which the depth should not be increased without some provision for taking the extra shear. In a subsequent article the writer hopes to show that this limiting depth is one-tenth of the span. When the depth of beam is more than one-tenth of the span, some provision must be made for taking the shear that the concrete is not capable of carrying. The mere turning up of some rods and ending them short of the end of span, or running them to the end of span without anchorage at the end of span, will not do this.

For columns a circular coil to resist the bursting or bulging pressure and longitudinal rods tied to this coil at regular intervals in the circle to resist flexure, seems to be the most rational means of overcoming the weaknesses of concrete as a column.

The Design of Reinforced Concrete Beams and Slabs.

Based on the principles of design laid down by the writer in an article published in *Concrete Engineering* of January 1, 1907, the following is given as exemplifying simplified, and at the same time rational design as applied to beams and slabs in reinforced concrete.

There are two ways of finding the safe strength of a beam or slab, one is to use in the formula ultimate values and then apply a factor of safety; the other is to use in the formula safe values, thus applying the factor of safety before the formula for strength is reached. These two methods should not be divorced from each other, but their interdependence should be recognized. Stresses that would be quite safe for steel in a framed structure may be unsafe in steel imbedded in concrete, by reason of the excessive stretch and the consequent cracking of the concrete. And again there should be some approximate agreement between the ultimate strength of beams as shown in the formula and the ultimate strength of test beams as found by experiment. Formulæ to be of value, must be based upon assumption of the continued elasticity of the materials up to the point of failure, and for this reason exact agreement between the calculated ultimate strength and that shown in tests is not to be expected. In fact the variation in the strength of concrete would preclude any such exact agreement.

The ultimate usefulness of steel is reached when the material is stressed to its elastic limit, or say 40,000 pounds per square inch, which is about the elastic limit, as shown by drop of beam, for the vast majority of tests commercially made on structural steel. High steel is not considered in this connection, for its use is neither appropriate nor economical. At this value for the stress in steel cracks are sure to be plainly visible and bond *between the concrete* and steel is greatly weakened, if *not destroyed. Using* 40,000 pounds per square inch a

the ultimate tensile value and a factor of safety of four for rolling loads, such as those on floors where wheeling is done, or in bridges, we have 10,000 pounds per square inch as the safe limit on the steel. It is safe to say that if the stress on the steel be kept within this limit, other parts of the design being equally well proportioned, the integrity of the structure will be completely safeguarded.

Unlike steel, concrete is elastic practically up to the point of failure. Good stone concrete, of the proportions of 1 volume of Portland cement to 2 of sand and 4 of hard broken stone, will show in short blocks an ultimate strength of about 2,000 pounds per square inch after setting several months. This may then be taken as the ultimate strength of concrete in beams, and on the same basis as the steel the safe value is 500 pounds per square inch. The ratio then between the unit stress in the steel and the extreme fibre stress in the concrete is 20.

By the well-known principle of the equality of tensile and compressive stresses in a beam subject to bending only we know that the total tension in the steel must equal the total compression in the concrete. As stated in the article previously referred to, no tension will be counted upon as existing in the concrete. The variation of stress in the concrete will be taken as uniform from the neutral axis up, and the neutral axis will be taken as located in the middle of the depth of the concrete beam or slab.

Let D=depth in inches of concrete beam or slab out to out; B=width of beam in inches; A=area of steel in square inches; M=ultimate bending moment in inch-pounds on beam AB; M'= bending moment in foot pounds on beam or slab one foot in width.

The ultimate stress in concrete is $2{,}000 \times \frac{1}{2}D \times \frac{1}{2}B =$ $500\ BD$. This must equal the stress in the steel or $40{,}000\ A$. Hence A=1-80 of BD or 1.25 per cent. of the area of the concrete rectangle.

The center of gravity of the stress in the concrete is

one-third of D above the neutral axis, and, if we place the steel one-eighth of D from the bottom of concrete, it will be three-eighths of D below the neutral axis. The effective depth is then 17-24 D, and the ultimate bending moment is

$$M = 17\text{-}24D \times 500BD = 354BD^2$$

If a beam or slab 12 inches wide be considered, the ultimate bending moment on the same in foot-pounds will be

$$M' = 1\text{-}12 \times 354 \times 12D^2 = 354D^2$$

Allowing a factor of safety of four for rolling loads and 3.5 for quiescent loads we have for safe bending moment per foot width

$$M' \text{ for rolling loads} = 88D^2 \ (1)$$
$$M' \text{ for quiescent loads} = 101D^2 \ (2)$$

By the same process we may derive the following for steel placed one-sixth of the depth from bottom of concrete

$$M' \text{ for rolling loads} = 83D^2 \ (3)$$
$$M' \text{ for quiescent loads} = 95D^2 \ (4)$$

For steel placed one-tenth of the depth the safe bending moment per foot width is

$$M' \text{ for rolling loads} = 92D^2 \ (5)$$
$$M' \text{ for quiescent loads} = 105D^2 \ (6)$$

The foregoing considers only the bending moment in the beam or slab, and deals only with the amount and location of steel to reinforce a concrete beam for a given bending moment. In all beams shear plays an important part and must be taken care of. In steel beams this office is performed by the web plate. In reinforced concrete beams provision for gripping the rods must be made. This corresponds in structural steel design to the spacing of rivets in the flange to take the flange increment.

Some of the rules laid down in the writer's previous article are these:

(a) The diameter of horizontal straight rods not anchored at the *ends of span* should not exceed 1-200 of the *span.*

(b) The spacing of rods in which the gripping value is taxed to the safe limit (as where the diameter is 1-200 of the span) should be four diameters for square rods and 3.1416 diameters for round rods.

Coupled with the latter rule it is to be observed that the distance from center of outside rod to edge of beam should be one-half of the spacing. This is so that the shearing area in a plane just above the rods will be equal to the superficial area of the rods. If rods of less diameter than 1-200 of the span are used, the gripping value is not taxed to its safe limit. In such case the spacing can be closer. The area of the horizontal section of the beam should be not less than the superficial area of the rods in a length of 200 diameters, for beams taking uniform loads.

Rods should be several times their diameter from the bottom of the beam, so as to make the gripping of concrete effective. With 1.25 per cent. of steel and round or square rods spaced 3.1416 and 4 times their diameters apart respectively, rods ⅛ of D from the bottom of concrete, it will be found that the center of rod is 2½ diameters from the bottom of the concrete beam. This is in good proportion.

By rules (a) and (b) the gripping of rods and the increment to the stress in rods, as well as the horizontal shearing strength of the beam, are taken care of.

For vertical shear, with only horizontal rods, the end shear must be taken by the rectangle of concrete having sides B and D. It is a well known principle of mechanics that the intensity of shear in a rectangular beam varies as the ordinates of a parabola, being a maximum at the middle of the depth of rectangle, where it is three-halves of the average.

Concrete in shear partakes of the weakness of concrete in tension, because it depends for its value to a large extent on the tenacity of the material. In simple compression a loose hard material, such as sand, if it be confined, has great strength, but it can have no shearing

strength. Reinforcement confines concrete to a large degree, and lessens dependence upon tenacity in the material itself. Concrete in compression can therefore have a much greater relative safe value than in shear. Shear must be depended upon in design, but its unit value should be low. If 50 pounds per square inch be taken as a safe limit in shear, the average on the rectangle should be two-thirds of this. The allowed end shear is then shown by the following equation

$$\frac{W}{2} = 12\,D \times 2\text{-}3 \times 50, \text{ or } W = 800\,D$$

where W is the total load in pounds on a beam or slab 12 inches wide.

The bending moment in a uniformly loaded beam, carrying a total load W is W times ⅛ of the span. If $L =$ span in feet we have, from (1)

$$M' = 88D^2 = 800D \times \frac{L}{8} = 100DL$$

or $88D = 100\,L$, and since D is in inches and L in feet, the actual ratio of D to L is about 1 to 10. Hence, when the depth of a beam or slab is greater than one-tenth of the span, the concrete is overtaxed in shear, and must be reinforced.

This reinforcement is best effected by curving up some of the rods and anchoring them at the ends or passing them over into adjacent spans.

By an arrangement such as shown in Fig. 1 the total end shear is taken by the rod, at least such of the shear as is represented by the load giving the stress in the rod that is curved up. If some of the rods are curved up and anchored at the ends of span and others are horizontal throughout, the concrete will be called upon to take only the end shear represented by the part of the load taken by the horizontal rods.

Continuous beams are of frequent occurrence in reinforced concrete being almost a necessary result of the

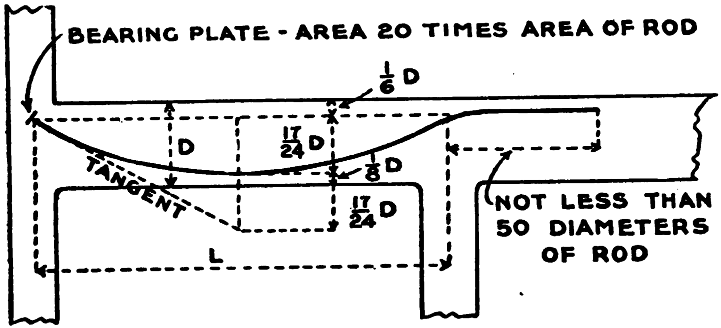
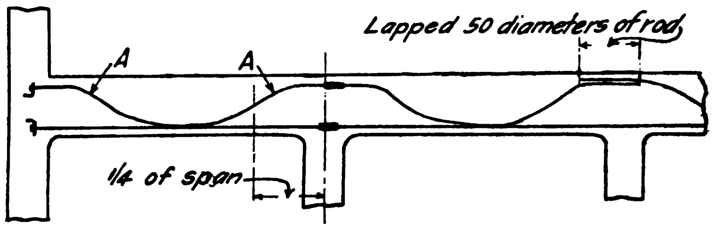


FIG. 1. SHOWING ARRANGEMENT BY WHICH TOTAL END SHEAR IS TAKEN BY THE ROD.



nature of its manufacture. The bending moment on an indefinite line of continuous beams all uniformly loaded, or that on a fixed ended beam, is two-thirds of that of a simple beam. This moment is negative and occurs at the supports; at the center of one of these beams the bending moment is less, but, if the adjacent beams be relieved of their live load, the bending moment at the center of this beam increases to an amount depending upon the relation between dead and live load. For these reasons some designers use two-thirds of the simple beam moment for reinforcement at center and ends of beams that are continuous. There are several objections to this. First, the formula for continuous beams is based on beams having a uniform moment of inertia. This is not true of reinforced concrete beams. Second, it is generally impracticable to make the last beam of a line fixed at the end, and the bending moment of a beam of uniform moment of inertia, fixed at one end and simply supported at the other, is equal to that of a beam simply supported at both ends. This bending moment is, however, at the fixed support and is negative. In a continuous beam the moment at this support may exceed two-thirds of that of a simple beam. Third, uniform load on all spans does not give the maximum moment at the middle of a span. Fourth, the relief moment at the supports in any beam under consideration, assuming that beam to be fully loaded and the side beams unloaded, is at the expense of negative moments in the side beams, which may require reinforcement in the top of beams throughout their length. Complete reinforcement at top and bottom of beams throughout the span is not desirable and is not economical. Fifth, some beams in the structure may not be joined end to end with other beams, and some may not be of the same span as those adjacent. Complications would thus be introduced that no simple rule would cover. Sixth, in alterations on the structure a beam may be taken out upon which another depends for its

strength. Seventh, unequal settlement of supports will disturb the condition of assumed continuity.

The first reason given above is sufficient to condemn the use of the purely theoretical method, not because the theory itself is incorrect, but because it is not applicable. Fully carried out the theory of the continuous beam would greatly complicate both the design and the execution, and it would not possess the redeeming feature of being correct. A further reason why top reinforcement as a primary element of strength should be avoided where practicable is this practical one. In a fire the principal damage to floors is on the under side where heat is greatest, and is usually, in the case of concrete, of the nature of spalling at the surface. Damage to the concrete of a beam or slab on the tension side affects only the grip on the steel, and would have to be extensive to be serious; whereas a small amount of spalling on the compression side would greatly weaken a reinforced concrete beam.

If each beam be reinforced for its full bending moment as a simple span in the way outlined above, there can be no question as to its safety, provided the top flange stress near the support is not such as to crack the concrete and thus weaken the beam in shear. There remains, then, the necessity, in beams where continuity exists, of preventing tension in the concrete in this portion of the beam. The rigidity of the beam, if fully reinforced for the simple beam moment, will be greater than if reinforced for only a part of that moment, and the deflection will be less. Hence the tendency to produce tension in the top flange near the supports is diminished. To overcome this tendency it is recommended that reinforcement be used to the amount of one-half of that at the center of beam and that the reinforcing rods extend at least through one-quarter of the span. This would be equivalent in total reinforcement to three-quarters of the full reinforcement at center of span and three-quarters at supports, and would be distributed in a more rational manner.

In an arrangement such as shown in Fig. 2, if the upper rod is made continuous and horizontal near ends of span for a portion of the span, as indicated, the tension in the upper part of the beam at supports will be taken care of. Tension in these rods, however, in the curves at *A* gives a force in the direction of the arrows, causing tension in the bottom of beam. In such case the bottom rod should also be continuous or else anchored at supports, as there is a tension under the arrows, where these rods would have developed only a portion of their anchorage, if not secured at the ends of span. With rods so arranged in continuous beams one-half of the reinforcing rods may be brought up near the top flange over supports and the other half made horizontal throughout. Splicing of rods should be over supports, either with sleeve nuts, which is preferable, or by lapping rods fifty diameters. The continuity of bottom rods, as well as top rods, adds greatly to the rigidity of the building.

In the case of slabs of short span there is scarcely any need of reinforcement near the top of slabs over supports, because of the extra concrete that is generally present. The concrete supplies a rigidity that does not allow of much deflection, and the danger of cracking, if the slab is properly reinforced near the bottom, is small. The writer made some tests on concrete slabs 3 feet 8 inches in clear between the beams and 3 inches thick, in which there was no reinforcement whatever. Many of these were cracked through the middle parallel to beams. In spite of the cracks and the absence of reinforcement the slabs showed no signs of distress under 250 lb. per square foot of superimposed load. Slabs of this sort should undoubtedly be reinforced, but owing to the excess of concrete made necessary by practical considerations of thickness, the rigidity resulting therefrom is warrant for the omission of top flange reinforcement.

Continuous slabs of long span deflecting as simple beams *would bend* at *sharper* angles over the support *than short span slabs.* Hence the need of top flange rein-

forcement in longer spans. Slabs in which there is not an excess of concrete over the requirements for compression will not be as rigid as those in which there is an excess of concrete. It is recommended that in slabs of a span of 9 feet or more, and in all cases where there is an area of three-quarters of one per cent or more of steel, one-half of the reinforcing rods be curved up over supports, as indicated in Fig. 2.

As an example of the application of the foregoing in the design of the floors of a building, given a building having columns 15 feet apart, to be designed for a live load of 100 pounds per square foot, assumed to be quiescent. Assume girders between columns and the beams supported by the same to be spaced 5 feet apart. For the floor slab assume a depth of 3 inches. By equation (4) the bending moment per foot of width is 855 ft-lb. The dead load in this case is 40 lb. and the live load 100 lb. per sq. ft. The total moment is then $140 \times 25 \div 8 =$ 438 ft.-lb. The area of steel reinforcement, instead of being $1\frac{1}{4}$ per cent of 12x3 sq. in. (or .45 sq. in.) will need to be only 438/855 of .45 or .23 sq. in. per ft. width of slab. This can be made up of $\frac{1}{4}$-in. square rods spaced 3 in. apart. These should be placed with centers $\frac{1}{2}$ in. above the bottom of slab. In addition there should be say two rods running parallel to the beam to tie the concrete together in that direction.

For the beams the span is 15 feet. The slab weighs 200 lb. per ft. of beam; the assumed weight of beam in addition is 130 lb. per ft. The total load is then 830 lb. per ft. and the bending moment is

$$830 \times 15^2 \div 8 = 23400 \text{ ft. lb.}$$

Assuming a depth of 18 in. we have by equation (2)

M' per foot width of beam $= 101 \times 18^2 = 32700$ ft.-lb. The width of beam is then $23{,}400 \div 32{,}700 = .716$ ft. or say $8\frac{1}{2}$ in. For reinforcement $1\frac{1}{4}$ per cent of $18 \times 8\frac{1}{2}$ $= 1.9$ sq. in. The allowed diameter of rods is 1-200 of 180 in., or .9 in. Three round rods of this diameter would make up the exact area required. Further, the circum-

ference of the three rods would just about equal the width of the beam. The nearest commercial sizes to this are ⅞-in. and 15-16-in. For the purpose of having an even number of rods four round rods 13-16-in. in diameter will be used. The superficial area of these rods in 200 diameters, (= 162.5 in.) is 1,658 sq. in.; the area of a horizontal section of the beam is 8½ × 180 = 1,530; but as these rods are of larger diameter than necessary, the horizontal shear would be taken care of in this beam if all rods were horizontal. To take the stresses due to continuity two of these rods will be curved up at supports, as shown in Fig. 2. At center of span all rods will be 2¼-in. from bottom of beam.

For the girder assume a depth of 24 in., including the depth of slab. The total dead weight of bay is about 19,000 lb., and the live load carried is 22,500 lb., a total of 41,500 lb.

When a girder carries beams equally spaced, two of which are at the ends of span, the bending moment is the same as though the load were uniformly distributed, if one of these beams is at the center of span, that is, if there is an even number of spaces. If $W=$ the total load, $L=$ span in feet, the bending moment $=WL \div 8$. If there is an odd number of spaces, the bending moment is less by the value of the moment that would occur in a span equal to the space between beams. For example, if there are five spaces, the bending moment in a span

$\frac{L}{5}$ is $\frac{W}{5} \times \frac{L}{5} \div 8 = \frac{WL}{200}$, and that in the girder is

$$\frac{WL}{8} - \frac{WL}{200} = \frac{3\,WL}{25}$$

In the present case there are three spaces, and the bending moment is

$$\frac{WL}{8} - \frac{WL}{9 \times 8} = \frac{WL}{9}$$

The bending moment on this girder is then

$$\frac{41{,}500 \times 15}{9} = 69{,}200$$

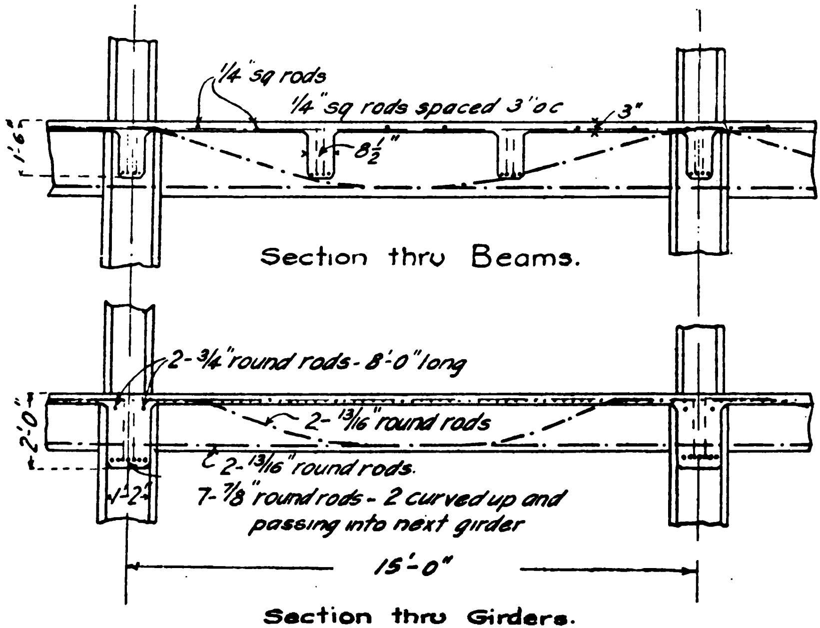

1/4" sq rods
1/4" sq rods spaced 3" o c
3"
8 1/2"
1'-6"
Section thru Beams.
2-3/4" round rods - 8'-0" long
2-13/16" round rods
2-13/16" round rods.
7-7/8" round rods - 2 curved up and passing into next girder
2'-0"
1'-2"
15'-0"
Section thru Girders.

FIG. 3.

By equation (6) $M' = 105D^2 = 105 \times 24^2 = 60480$ ft.-lb. per ft. width of beam. The beam should then be 14 in. wide. The area of steel required is 1¼ per cent of $14 \times 24 = 4.2$ sq. in. This may be made up of seven ⅞-in. round rods, five of which lie horizontal and two of which are curved up as in Fig. 1. In addition to these two ⅞-in. rods, which pass into adjacent beams to take the stress due to continuity, there can be two ¾-in. round rods 8 ft. long placed near the top of girders over the support, to make up the remainder of the 50 per cent of steel area.

The end shear of this girder is about one-third of the load of a bay or 14,000 lb., as the load of one beam is taken directly to the column. At a nominal ultimate of 200 lb. per sq. in. on concrete in shear, with a factor of safety of 3½, the allowed unit is 57 lb. per sq. in. Two-thirds of this as the average on the rectangle = 38 lb. per sq. in. This on a section 24 in. x 14 in. gives 12,800 as the shear allowed on the concrete. As the concrete is called upon to take only 5-7 of the shear the shearing strength is ample. At center of span all rods will be 2.4 in. from bottom of beam (say 2½ in.). The five horizontal rods are to be equally spaced, being 2¾ in. center to center, the outside ones being 1½ in. from the surface. The two curved rods may lie between horizontal rods. Fig. 3 illustrates this floor.

Reinforced concrete lends itself to construction in the form of slabs supported on four sides, and such construction may in some cases be more economical than the ordinary beam and slab design. Given a square slab supported on all four edges, the side of the square being L feet, as shown in Fig. 4. Assume that this slab is to be reinforced in two directions at right angles to each other. It is clear that the strip A B, 1 ft. wide, and the strip *1* each being similarly placed in the structure will each sustain the same load. Each will then take one-half *of the load on the middle* square foot of the slab. The *strips 2, 3, 4, etc., will* be prevented from deflecting as much

as strip *1* and hence will take less load than strip *1* in proportion as their deflection is less. Strip A B will then be compelled to carry more than half of the load on the square at the intersection of itself and strip *2* and still more at the intersection with strip 3, etc. It is sufficiently accurate to assume that the deflections of strips 1, 2, 3, 4 etc., diminish as the ordinates of a parabola. Hence the load on strip A B increases in the same relation. Fig. 4, at (b), illustrates the loading on strip A B. The bending moment at the center of span for the uniform load of intensity $\frac{1}{2}$ w, is $wL^2 \div 16$. For the load increasing from zero at middle of span to $\frac{1}{2}w$ at the ends, it can be shown that the bending moment at the middle of span is $w\,L^2 \div 96$. The total moment at center of span

is then $M' = \frac{w\,L^2}{96} + \frac{w\,L^2}{16} = \frac{7\,w\,L^2}{96}$ (7)

This strip is the critical strip of the slab. Reinforcement can be diminished towards the beams surrounding the slab. This diminution is not uniform, however, but as the ordinates of a parabola. At a point $\frac{1}{4}L$ from the beam the reinforcement must be not less than $\frac{3}{4}$ of that at the center. A good rule is to use the same reinforcement for the middle half of the span and uniform variation from the quarter points to a minimum at supports.

For the beams supporting this slab an intensity of load may be assumed equal to the reaction of strip A B, or 1-3 w L. Assuming a uniform load on the beams we find

$$Moment = \tfrac{1}{3}\,w\,L \times \frac{L^2}{8} = \frac{w\,L^3}{24} \quad (8)$$

If the beam supports two equal square slabs, *moment* =

$$\frac{w\,L^3}{12} \quad (9)$$

It is to be noted that this is the same moment as would be obtained by assuming that each beam on the side of the square supports a load uniformly increasing from ends to middle of span, being $\frac{1}{2}$ wL at middle for *each square* of slab supported. In using this formula w *should be* taken as the weight per sq. ft of slab plus the

uniform live load per sq. ft. The weight of the beam itself below the slab is a uniform load whose moment is found in the ordinary way.

Exactness is not claimed for this theory. It is based on reasonable assumptions, however.

On account of the fact that the cross rods in the slab cannot occupy the same position the reinforcement must be proportioned accordingly.

In a rectangular slab supported on four sides and oblong in shape the amount of reinforcement needed in the two directions is found as shown in Fig. 5*a*. It will be seen by trial that when there is much difference between the sides of the oblong the short span will take nearly all of the load. Thus when one side is 50 per cent longer than the other $k = 1$, showing that in an oblong slab of these proportions reinforcement in the long direction needs to be only nominal.

As an illustration of the application of the above to a floor in square slabs, take the case of columns spaced 15 feet each way, live load 100 lb. per sq. ft. Assuming a weight of slab of 70 lb. per sq. ft. we have by equation (7) a total moment on one foot width of slab = 2,790 ft.-lb. By equation (4) we find a depth of slab of about 5½ in. One and one-quarter per cent of 3 in. x 5½ in. would require just a little more than 7-16 in. square rods. This size of rod could be used, spaced 3 in. apart and ⅞ in. above the bottom of slab, but this would not leave room below for the layer of rods in the other direction, with sufficient concrete. To meet the difficulty we will add ¼ in. to the depth of slab and make the plane separating the two layers of rods one inch from the bottom of slab. The spacing of 3 in. will be used for the middle seven feet, increasing to 12 in. towards supporting beams. To take care of the continuity every alternate rod in the middle half of slab will be brought up near the top of slab at quarter points, all other rods being straight. This will make *a sort of square* basin in the middle of the slab.

Assume a beam supporting the slabs of a depth of 24

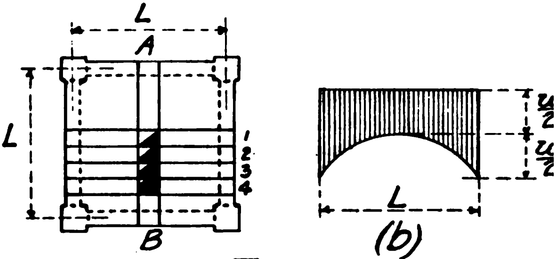
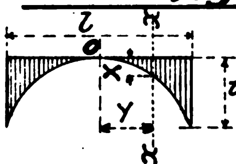


Fig. 4.



$$M \text{ at } x\cdot x = \frac{wl^2}{48} - \frac{w}{3}$$

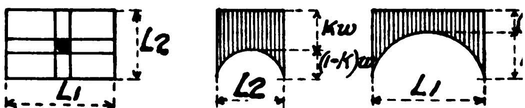
$$\frac{d^2x}{dy^2} = \left(\frac{wl^2}{48} - \frac{wy^4}{3l^2}\right)$$

Equation of elastic line (Origin at

$$X = \left(\frac{wl^2y^2}{96} - \frac{wy^6}{90l^2}\right)\frac{1}{EI}$$

$$\text{Deflection at } o = \frac{7wl^4}{2880EI}$$

Fig. 5.



For equal deflections

$$\frac{5KwL_2^4}{384EI} + \frac{7(1-K)wL_2^4}{2880EI} = \frac{5(1-K)wL_1^4}{384EI} + \frac{7Kw}{288}$$

$$\text{solving} \quad K = \frac{75L_1^4 - 14L_2^4}{61L_1^4 + 61L_2^4}$$

$$\text{Moment on span } L_2 = \frac{KwL_2^2}{8} + \frac{(1-K)wL_2^2}{48}$$

Fig. 5a.

in., including slab, and a weight of 200 lb. per ft. below the slab. By equation (9) the total moment from the slab is 47,800 ft.-lb. Adding 5,630 ft.-lb., due to weight of beam, we have a total of 53,430 ft.-lb. By equation (6) we find the width of beam to be 10½ in. The reinforcement can be made up of four ⅞-in. square rods, two of which curve up, as in Fig. 1, and pass into adjacent spans. At the middle of span all rods are 2½ in. from bottom of beam.

Comparing this floor with the one worked out above having slab, beams, and girders, we find that the one having beams and girders contains about 124 cu. ft. of concrete per bay or square, while the other contains about 148 cu. ft. The forms would be simpler in the square slab construction, and this would tend to balance the cost.

A reinforced concrete building should be tied together as a unit, just as a building having a steel frame is tied together by the rivets that connect the beams to columns. This can best be effected by making rods in beams and slabs continuous, either by connecting them end to end with turnbuckles or by lapping them over supports. It is hard to see how a failure of any considerable extent could take place in a building so tied together, even under the severest conditions, assuming that the columns are properly designed.

In the execution of plans for reinforced concrete it is very essential to have steel placed just where calculations show that it should be placed. This is not always easy to do, especially in any complicated design. Permanent templates left in the concrete may work an injury by making a plane of cleavage for the starting of a crack. Exactness in placing the steel may be secured in the following manner:

Given a beam having reinforcing rods placed as per Fig. 6, at any given section. A template of wood is made fitting the cross section of the beam and extending up above the slab 6 or 8 in. Holes are bored in this template for each of the reinforcing rods. It is then sawed through

these holes into pieces that can be turned and removed from the box which forms the mold for the beam. It could, of course, be made of several pieces fitted together. When in use the parts are clamped together as in the figure, with the rods passing through the holes. Enough of these templates should be used to hold the rods in position during placing of the concrete. When the concrete has been placed nearly to the template on both sides, the rods will then be held by the concrete itself. The templates may then be unclamped and the pieces turned and drawn out, and the concreting completed.

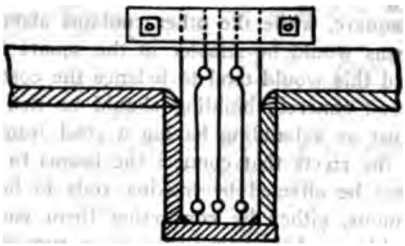


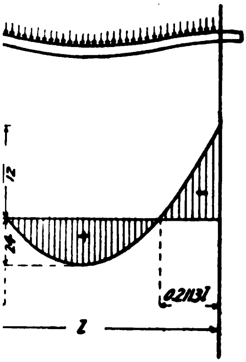
FIG. 6.

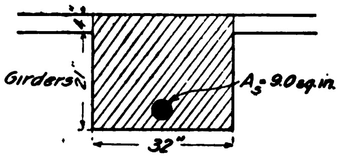
DISCUSSION OF T BEAMS AND SLABS.

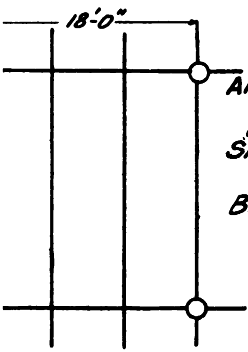
Editor Concrete Engineering:

The article by Mr. Edward Godfrey in your issue of Jan. 1, was an excellent one, and if the many good recommendations which it contained were generally followed the number of failures in reinforced concrete constructions would no doubt be greatly reduced, if not wholly eliminated. Reinforced concrete has lately received a bad reputation, and any rules and recommendations which tend to spread knowledge on the subject, and influence designers and superintendents to exercise better judgment and care in the execution of their work, are very welcome.

Referring to the clause in the article dealing with T beams, the writer, although admitting that it is on the *side of safety*, believes that it is unnecessary extravagance to figure beams, in the way recommended by Mr. God-





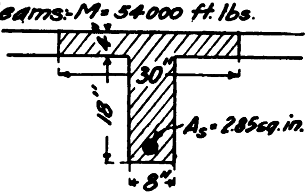


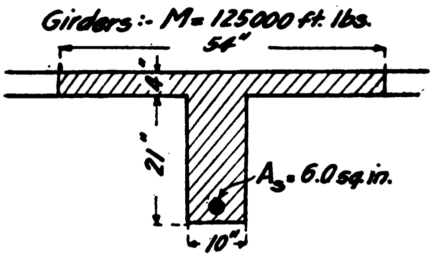
Live floor load = 120 lbs/sq. ft.

Alternative I :- Part of the slab is included in the beam calculation.

Slab :- M = 645 ft. lbs.
4" thick slab - Steel = 1.0 lb/sq. ft.

Beams:- M = 54000 ft. lbs.



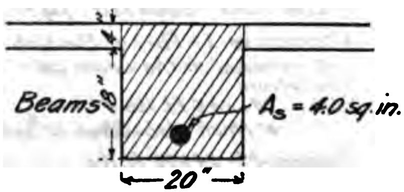


Concrete.	Steel.	
Slab.........0.33	1.0	cu. ft. 2 lbs./sq. ft. of floor.
Beams.....0.167	2.0	" "
Girders ...0.073	1.27	" "
Total=0.570	4.27	" "

ALT. II. FLOOR SLAB DISREGARDED IN CALCULATING BEAMS AND GIRDERS.

RESPECTIVE BENDING MOMENTS AND DEPTHS OF FLOOR MEMBERS REMAIN THE SAME AS FOR ALT. I.

SLAB: REMAINS THE SAME AS FOR ALT. I.



Concrete.	Steel.	
Slab.........0.33	1.0	cu. ft. 2 lbs./sq. ft. floor.
Beams.....0.417	2.72	"
Girders ...0.233	1.83	"
Total=0.980	5.55	"

ALT. II IS CONSEQUENTLY 13.9 CENTS MORE EXPENSIVE PER SQ. FT. OF FLOOR THAN ALT. I—OR ABOUT 50%.

THE COST OF CENTERING WOULD ALSO BE IN FAVOR OF ALT. I.

rey. It is safe to say that in more than 50 per cent of all he reinforced concrete structures built in this country, nd in Europe, the beams have been treated as T beams, e., a portion of the floor slab has been considered as orming part of the beam, in calculating the strength of he same.

The fact that nearly all of these structures are standing oday and carry the loads for which they were designed, s at least partial proof that this method of calculation results in a sufficiently safe design.

The question of how much of the slab may be safely considered as part of the beam is a matter which should be determined by tests. The writer usually assumes that about 1-5 of the span on either side of the beam acts as top flange. This assumption is based on the condition that the floor slab is built monolithic with the beams and girders, and that the slab reinforcement is carried across the beams continuously, and tied down into the beam reinforcement, by means of stirrups or hangers. Such a slab may be considered as a practically built-in-beam, in which case the point of contra flexure occurs at a distance of (0.21 l) from the beam, for uniform loading. (See illustration.) Within this distance the upper part of the slab is in tension, and the lower part in compression, due to negative bending moments in the slab. In addition to these stresses the bending of the beam produces compressive stresses in the slab at right angles to the former. The latter have their maximum intensity at the top and tend to increase the tensile stresses, which are resisted by the slab reinforcements, and are practically nil at the bottom. It is often found impracticable to raise the slab reinforcement over and adjacent to the beams, so as to fully resist the tensile stresses, in which case, it s necessary to introduce small secondary rods of the required section and length near the top of the slab, as the concreting proceeds.

That a considerable economy is effected by considering *e beams of T section,* as compared with rectangular sec-

tion in designing, is apparent from the example given below, which, it is believed, represents an average case. The calculations of the beams and slabs are based on the regulations governing the design of reinforced concrete work in New York City.

Apart from the matter of economy, the beams which are figured as rectangular become very clumsy, and would be objected to in many buildings on that account alone.

A. JORDAHL.

Editor Concrete Engineering:

Sir: The letter from Mr. A. Jordahl in your issue of Feb. 15, in favor of the T-beam calculation of reinforced concrete, seems to me to prove something that is manifest and to pass without comment the issue brought up by my article in the issue of Jan. 1. I do not claim that it is cheaper to neglect the floor slab in calculation of the beams. If the concrete is needed around the rods to grip them and to take up the effect of differential expansion, it should be there for that purpose, and means should not be sought, through some hokus pokus of methods of calculation, to leave it out for the sake of cheapness or supposed economy. I do not mean by this to question the regularity of Mr. Jordahl's calculation. I have not checked his figures, but take them to be right according to some method or other of calculation. What I wish to say is that any method of calculation that requires in a girder having 426 sq. in. of concrete, 6 sq. in. of steel, and in another of the same depth, having 800 sq. in. of concrete, 9 sq. in. of steel to perform the same work, is radically wrong.

According to this method of calculation a beam of a *certain depth* can have nearly half of its concrete shaved off and 1-3 of its steel without affecting its strength; or conversely, if nearly 100 per cent be added to the concrete of a beam, leaving the steel the same, it will be *weakened* one-third on account of being shy that amount *in tensile* strength of the steel. This difference is d

ourse due to the shifting of position that the neutral xis is supposed to undergo. The method used by Mr. ordahl to locate the neutral axis is purely theoretical, nd it is theoretical in a dangerous way, in that it leaves ut of the account the one most important consideration, amely, the effect of the shrinking of the concrete in etting. Furthermore, tests do not in any way substaniate the results of this theory.

A fault in the girder preferred by Mr. Jordahl is, that e has 6 sq. in. of steel in a girder only 10 in. wide. If his were made up of 6 1-in. sq. rods, there would be only -3 of an inch between them. It does not seem possible or the concrete to grip effectually this amount of steel nd to overcome the effect of differential expansion due temperature changes. I have seen beams 3 in. wide ith 1⅛-in. sq. rods in them cracked up, under the dead ad alone, in a way that tended to prejudice one against einforced concrete construction. Under load the beams esembled a string of beads.

To reiterate, my position is this: If the neutral axis e taken at the center of depth of the concrete beam, here tests have shown it to be, the only saving in a -beam over a rectangular beam is in concrete in the wer part of the rectangle, and the concrete is needed ere to protect and grip the steel. The steel in each case ould be exactly the same in amount.

EDWARD GODFREY.

FLOOR SLAB CONTROVERSY CONTINUED.

ditor Concrete Engineering:

Sir: Permit me to say a few words in reply to Mr. odfrey's letter in your last issue. My letter in your sue of February 15th, merely attempted to show that nder certain specified conditions it is entirely safe to inlude part of the floor slab in calculating the strength of eams, and *that such procedure* is attended with consid-*able economy. I did not* pretend that the (T) beams in e *example submitted* were as strong as those of rec-

[illegible] but I did claim that they were [illegible] [illegible] work, and that the rectangular bea[m] [illegible] [illegible] waste of materials, due to the [illegible] [illegible] based on the extravagant assumpt[ion] [illegible] did not add to their strength.

[illegible] interesting to have Mr. Godfrey furnish [illegible] [illegible] concrete T beams—which uphol[d] [illegible] the neutral axis is at the center of the [illegible] — [illegible] percentage of reinforcement.

[illegible] Mr. Godfrey sees in my girder, in the [illegible] a width of 10 in., is not as seri[ous] [illegible] The [illegible] in. sq. bars would be placed in [illegible] the underside of the first row $1\frac{1}{2}$ in. fro[m] [illegible] with a clearance of $1\frac{1}{2}$ in. between the [illegible] The bars in each row would b[e] [illegible] clearance and $1\frac{1}{2}$ in. from the sides [illegible] [illegible] the steel being placed in its corre[ct] [illegible] the beam box, and held there while the co[ncrete] [illegible] necessary to make it up into fram[es] [illegible] saddles and spacers, and for the [illegible] to place the steel in 2 be[ams] [illegible] there are only 4 rods in the bea[m]. [illegible] the effective depth of the beam [illegible] more than offset by the increased securi[ty] [illegible] handling and placing the steel.

[illegible] I have seen a large number of concre[te] beams [illegible] designed as T beams and carryi[ng] [illegible] several times that for which it was de[signed] [illegible] without showing any cracks or other evidence [of] weakness.

A. Jordahl

Editor Concrete Engineering:

Sir: Replying to Mr. Jordahl's letter, in rebuttal [I] would state briefly that if he did not pretend that the [T] beams in his comparison were as strong as the rectangul[ar]

beams, why did he compare them in cost in order to discredit the rectangular beam?

I cannot give any data of tests on T beams that will prove that the neutral axis is at the center of the depth. Can Mr. Jordahl give any to prove that it is not? Mr. Jordahl's theory is entirely upset in the case of rectangular beams by tests which show the neutral axis close to the center for safe loads. Why should it be held to be good in T beams? The modulus of elasticity of concrete varies all the way from one million to five million. Now if such a peripatetic value as this is to have any weight in designing and calculating beams, where are we at? The strength of a beam would be a function of the designer's private opinion as to what the modulus of elasticity ought to be.

EDWARD GODFREY.

The Design of Reinforced Concrete Arches.

The arch is about the least promising form of reinforced concrete design to standardize by reason of the apparently refractory facts about an arch and its loading. In any investigation of the stresses or the stability of an arch assumptions must be made, since the same degree of accuracy is not possible in arches as that which obtains in steel frames or even in reinforced concrete beams. The premises and assumptions upon which the following investigation is based are these:

(1) Concrete will be considered to take only compression (and the shear incident to the joint action of concrete and steel as a beam to resist bending). The unit compression, or the extreme fibre stress on concrete will be taken at 500 lbs. per square inch. This would not be a safe value on a plain concrete arch, but by virtue of the steel reinforcement, which ties the concrete together and relieves it *of bending* strains, the concrete thus held is *capable of withstanding* with safety a compressive stress *of this intensity.*

(2) Steel will be considered to take only tension. The unit tension allowed will be taken at 10,000 lbs. per square inch, because at this stress, it is believed, few if any cracks will develop due to the extension of the steel under tension.

(3) Earth will be considered to exert only vertical force. It is of course true that earth has the ability, under certain conditions, to exert horizontal pressure; but it is not good engineering to depend for stability upon earth exerting an active horizontal pressure, unless the conditions are such that a measurable deflection or movement horizontally, is not injurious. The compressibility of earth is well known. Buildings having their foundations in earth will settle gradually and often continuously. If an arch is made to depend for its stability upon earth against which it presses horizontally, there will be the same tendency of the earth to settle back in the direction of the pressure, and this with a resistance but a fraction of that of a horizontal surface of earth against a vertical load. An assumption of horizontal pressure of the earth fill over an arch gives a curve of stability for the arch that is more advantageous and would give a more economical arch, but the nature of the case does not warrant this assumption.

(4) An arch will be considered stable and safe if the arch ring is capable of resisting the thrust and bending moment due to any possible applied loading, when the curve or polygon of equilibrium is drawn in such position as to give a minimum bending moment, or minimum deviation from the central line of the arch ring. In the case of stone or plain concrete arches it is conceded that the arch will be stable if an equilibrium polygon can be drawn, using any possible applied loading, that will pass within the middle third of all joints. The two propositions are equally sound.

(5) The arch proper will be taken as the part of the structure included between two vertical planes through the inner faces of the arch abutments. The rise of the arch will be taken as the middle ordinate of the central line of

the arch ring, considering the curve to terminate where it pierces the planes above referred to. If an arch be designed with a curve at springing joining the intrados and the inner face of abutment wall, as will often be the case for appearance sake, this small curve should have no structural significance. The effective rise of the arch is only obscured by considering the curve as having any weight in determining the rise. The intrados of the arch, the extrados, and the median line between these (the central line of the arch ring) will in general be close approximations to arcs of circles, each having one radius. Embellishments on the intrados, for grace of outline, should be considered as such and omitted from the calculations.

(6) The abutments of the arch will be separately treated as abutments and not as part of the arch. Their office as anchoring mediums for the reinforcing rods of the arch will also be recognized.

(7) It is understood that all calculations refer to a foot in width of the arch ring and that the arch is one having fill in the spandrel up to the floor level. For a ribbed arch a modification can be made by using factors that will represent the weight carried by one foot in width of the rib. The arch will be taken as hinged at the abutments.

Fig. 1 gives a typical form of arch. The intrados, extrados, and median line are here shown as arcs of circles. As stated these curves will approximate circular curves; the actual curve to be used must be determined by the calculations. Attention is called to the method of finding the thickness of arch ring. If the intercepts marked "equal" in the figure are so made, the thickness of arch ring will be proportional to the secant of its inclination to the horizontal, as should be the case where the thrust of arch is constant. A constant thrust necessarily follows where only vertical loads are considered.

In what follows earth will be assumed to weigh 100 lbs. per cu. ft. and concrete 150 lbs.

For a uniform live load of W lbs. per sq. ft. the thi

ness at the crown may be found in the following manner. The loads to be considered are (1) the arch ring, (2) fill from arch ring up to level of crown, (3) fill from level of crown to floor level. (This may include paving, by making proper allowance for weight in value of H), (4) live load. If we find the bending moment for these several loads at the center of span and divide the same by R, the result will be the thrust at crown. The allowed average stress on the cross section of the arch will be taken at 250 lbs. per sq. in., which would be the average with the extreme fibre stress 500 lbs. per sq. in. and intensity diminishing to zero at opposite edge of rectangle. A small deviation of the curve of equilibrium will throw the same one-sixth of the depth out of center, which would give the conditions just named.

The bending moment due to the fill from level of crown of arch down (assuming the curve of arch a parabola and remembering that the area of the fill to one side of the center is 1-6 S R and its center of gravity is 3/8 S from the center line of the arch) is

$$\frac{100S\ R}{6}\left(\frac{S}{2}-\frac{3S}{8}\right)=100\frac{S^2\ R}{48}$$

Now taking the effect of the arch ring as that of a uniform load 10 per cent greater than the depth of ring at crown we have for the total moment at center of span

$$M=\frac{100\ S^2\ R}{48}+\frac{WS^2}{8}+\frac{100\ H\ S^2}{8}+\frac{165\ D\ S^2}{8}$$

Dividing the right side of this equation by R we have the thrust, which we have assumed to equal $250 \times 144D$ (taking all dimensions in feet.) Hence we have

$$36{,}000D=\frac{25S^2}{12}+\frac{WS^2}{8R}+\frac{25HS^2}{2R}+\frac{165DS^2}{8R}$$

From this we may derive the following value for the depth of arch ring:

$$D=(2.08R+.125\,W+12.5H)\div\left(\frac{36{,}000R}{S^2}-20.6\right)\ (1)$$

This formula is to be used only for a trial depth. As

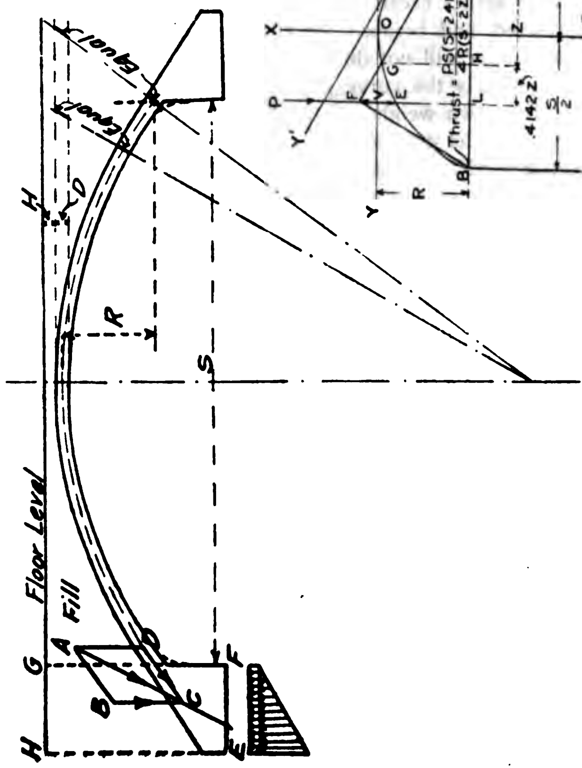

Floor Level
Fill
G
H
A
B
C
D
E
F
R
S

FIG. 1.

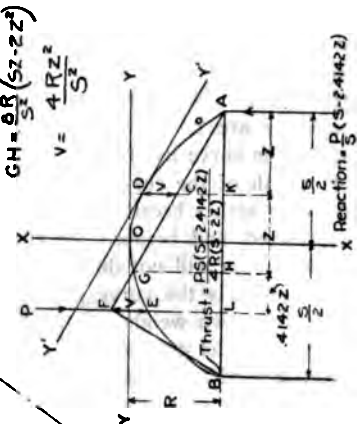

GH = 8R/S² (SZ-2Z²)
V = 4RZ²/S²
Thrust = PS(S-2.4142Z)/4R(S-2Z)
Reaction = P/S (S-2.4142Z)
.4142Z
S/2
Z
R
P
X
Y
Y'
A
B
C
D
E
F
G
H
K
L
O
V

FIG. 2.

will be shown later the effect of the live load covering a portion of the span is to produce an extreme fibre stress on the concrete greater than the unit stress from the thrust of arch fully loaded with live load.

A little study of some of the curves of equilibrium will be useful in the investigation of the arch.

A circular arc is the curve of equilibrium for forces normal to the curve itself and of equal intensity for every unit of length of the curve. This curve is useful in connection with arches because of the ease with which it can be constructed, and because the curve of equilibrium for an ordinary arch will not differ much from a circular arc.

The catenary is the curve that a flexible cord will assume from its own weight. It is the curve of equilibrium for forces parallel to each other and of equal intensity for every unit of length of the curve. The curve of equilibrium for an arch ring of uniform thickness, supporting the ring alone, is a catenary.

The common parabola is the curve of equilibrium for forces parallel to each other and of equal intensity for every unit along the chord of the curve. The curve of equilibrium for a uniform load over the entire arch, considering this load alone, is a parabola. The fill over the arch between floor line and a line parallel to the same through the crown would demand a curve of the same shape.

Another curve is one that would support a load represented by the distance from a horizontal line through the crown of the arch and the arch ring. Such a curve would be the curve of equilibrium for the fill over the arch below the level of the crown. Assuming the arch ring of an arch having a rise R and a span S to be a parabola whose equation is

$$y^2 = \frac{S^2\, x'}{4R}$$

we have for the conditions in a curve having the same *rise and span*, which would be in equilibrium under the ··r *this* parabolic arch,

$$\frac{dy}{dx}=\frac{thrust\ at\ (x, y)}{shear\ at\ (x, y)}$$

The thrust at any point is the same, and, as we have seen above, amounts to $\frac{S^2}{48}$ for a load of unity per unit of area of fill.

The shear at (x, y) is 1-3 x' y. (Note that x' is the absissa of the parabola and x of the curve under discussion, y corresponding for both).

$$\text{But } x'=\frac{4Ry^2}{S^2}$$

$$\text{Hence} \frac{dy}{dx}=\frac{S^2}{48}\times\frac{3\ S^2}{4Ry^2}=\frac{S^4}{64Ry^3}$$

$$\text{Integrating we have } y^4=\frac{S^4X}{16R} (2.)$$

This is a parabola of the fourth degree.

The catenary, the common parabola, and the curve whose equation has just been deduced will each have its influence in determining the shape of the curve of equilibrium, and hence the line of the arch ring itself, in an arch having fill over the haunches and supporting a uniform live load. The curve will be a sort of composite of all.

Using the circular arc and the parabola as standards by which to compare the other two we find that if all four curves pass through the same points at ends and crown, the catenary will lie between the circular arc and the parabola. For ordinary ratios of span and length it will be almost midway between these curves, being a little nearer to the parabola than to the circular arc. The circular arc will of course be outside of the parabola. The parabola of the fourth degree referred to will be outside of the circular arc. It will be flatter at the crown than any of the others and will rise above the circular arc over the haunches of arch.

The effect of the fill in the spandrel of the arch is to make the curve of equilibrium to rise above the circular arc at the haunches, and that of the weight of the arch ring and of the fill from level of crown to floor line, as

[illegible] dead load is to make the [illegible] [illegible]. The combined effect [illegible] circular arc, depending [illegible] coincidence of these two [illegible] the graphic method [illegible] line is made and [illegible] by combining [illegible] reactions, thrusts, [illegible] an arch of a given [illegible] as for a simple beam [illegible] the moment obtained by [illegible]. The thrust laid out to scale [illegible] representing the [illegible] will give the exact location [illegible] polygon pass through [illegible] arch at crown and springing. [illegible] method employed. The [illegible] section is equal to the [illegible] ordinate of the equilibrium [illegible] moment by the thrust, [illegible] ordinate of the polygon at the [illegible] ordinate for the center [illegible]

[illegible] approximate dimensions of the arch [illegible] simple beam moment are readily [illegible] arch should be drawn to scale, using [illegible] arc for the curve, and divided into vertica[l] [illegible] and the areas of these strip[s] [illegible] used to determine the applied loads. Then treat [the] arch as a simple beam or girder span the bending m[oment] may be found at center of span, which divided [by the] rise will give the thrust. Then the simple beam m[oment] at any other section may likewise be computed; d[ividing] this by the thrust will give the proper ordinate f[or the] arch.

It is believed the following method will give res[ults in] any ordinary case, that are commensurate in accurac[y with] the common assumption of the weights of concre[te]

th. The uniform live load, the fill above crown, and the total weight of arch ring can be taken as a uniform load over the arch. To find the bending moment at any section assume one-half of this total weight as concentrated at that section, treating the arch as a simple beam. The moment due to weight of fill below crown level is

$$M=100\left(\frac{R\,S^2}{48}-\frac{Ry^4}{3\,S^2}\right)(3)$$

Where $y=$ distance from center of span to section considered. This is the moment for a parabolic curve for top of arch ring.

Having found several points in the arch ring the curve can be drawn through these points.

The effect of live load on part of the span in altering the shape of the equilibrium curve is another important point to consider. This can be best seen experimentally by taking a cord already loaded, or a heavy cable, and suspending loads upon it. The added load will cause the cord to dip in the vicinity of the load. In general there will be a point on each side of the load, in the new curve, that will coincide with a point in the original curve, while the part between these points and the end supports will rise. By adjusting the length of the cord one of these points could be brought to any selected position. In the treatment of the arch the curve of equilibrium takes the place of the cord in the suspended system and responds to the shifting of the live load, rising where the other would dip, and vice versa. It is plain that any number of assumptions can be made as to the nodes or points in this curve which retain their original position. In accordance with the fourth of the premises given at the beginning of this article the node will be taken at such position as to make the curve of equilibrium to dip at one side of this node by the same amount as it rises at the other side, measured vertically.

By assuming that the curve of equilibrium for dead and live load is the same as that for the dead load alone, and that the two agree with the center line of arch ring,

analytic method to detegmine reactions,
the horizontal thrust of an arch of a
found by taking moments as for a simp
ter of span: dividing the moment ol
will give the thrust. This thrust laid o
the middle point of a vertical represent
applied vertical loads will give the ex
pole to make the equilibrium polygon
central line of the arch at crown an
avoids the usual cut and try method en
ple beam moment at any other section is
uct of the thrust and the ordinate of th
gon. Hence dividing this moment by
found, will give the ordinate of the pol
under consideration, or the ordinate f
of the arch.

Having the approximate dimensions c
culations for the simple beam moment
Of course the arch could be drawn to
circular arc for the curve, and divided
of any desired width, and the areas o
be used to determine the applied loads.

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By assuming that the curve of equilibrium for dead and live load is the same as that for the dead load alone, and that the two agree with the center line of arch ring,

well as the uniform live load, is to make the curve drop below the circular arc. The combined effect is to make the curve approach a circular arc, depending for its location upon the predominence of these two influences.

In treatment of arches the graphic method is generally used. This method used alone is crude and inexact. Much of its inaccuracy can be overcome by combining with it the analytic method to determine reactions, thrusts, etc. Thus the horizontal thrust of an arch of a given rise can be found by taking moments as for a simple beam at the center of span: dividing the moment obtained by the rise will give the thrust. This thrust laid out to scale opposite the middle point of a vertical representing the consecutive applied vertical loads will give the exact location of the pole to make the equilibrium polygon pass through the central line of the arch at crown and springing. This avoids the usual cut and try method employed. The simple beam moment at any other section is equal to the product of the thrust and the ordinate of the equilibrium polygon. Hence dividing this moment by the thrust, already found, will give the ordinate of the polygon at the section under consideration, or the ordinate for the central line of the arch.

Having the approximate dimensions of the arch the calculations for the simple beam moment are readily made. Of course the arch could be drawn to scale, using say a circular arc for the curve, and divided into vertical strips of any desired width, and the areas of these strips could be used to determine the applied loads. Then treating the arch as a simple beam or girder span the bending moment may be found at center of span, which divided by the rise will give the thrust. Then the simple beam moment at any other section may likewise be computed; dividing this by the thrust will give the proper ordinate for the arch.

It is believed the following method will give results, in *any ordinary* case, that are commensurate in accuracy with *the common* assumption of the weights of concrete and

earth. The uniform live load, the fill above crown, and the total weight of arch ring can be taken as a uniform load over the arch. To find the bending moment at any section assume one-half of this total weight as concentrated at that section, treating the arch as a simple beam. The moment due to weight of fill below crown level is

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By assuming *that* the curve of equilibrium for dead *and live load is the* same as that for the dead load alone, *and that the two* agree with the center line of arch ring

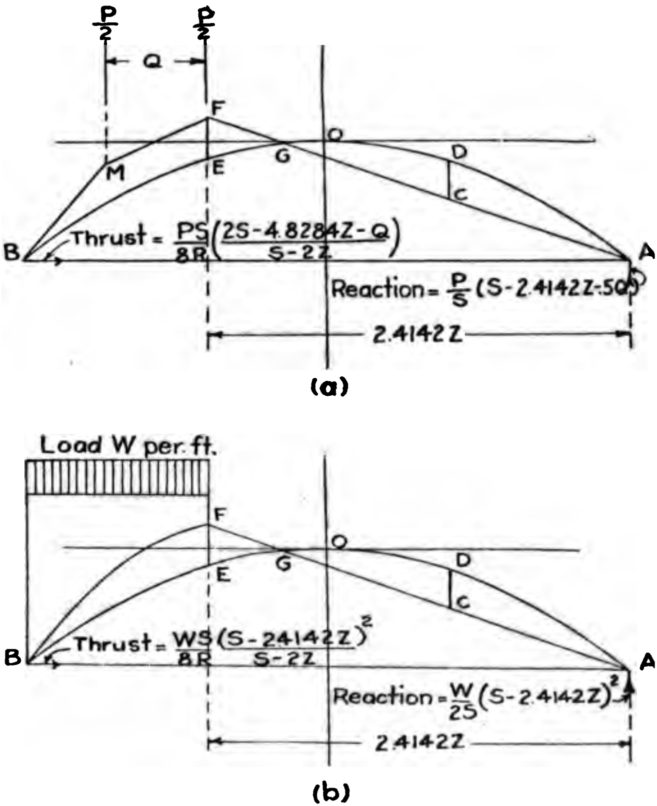
also that the curve is a parabola, the bending moment in the arch can be determined analytically in a manner to be shown presently. It is not meant that results obtained by assumptions that appear to be so loose as these are to be used in the final design of the arch. The assumptions afford a means of determining the position of live loads for the maximum effect on the arch, and supply a simple formula expressing this effect, the results of which are very close to the true results. The proper relation between span and rise, depth of arch ring and reinforcement can be studied by this means to an extent not approached by any graphical or other trial method.

If the curve of equilibrium for dead load coincides with the central line of arch ring, bending on the arch ring is eliminated excepting for live load, and we are left free to investigate the effect of a concentrated load or uniform load covering part of the span with the arch as a simple curved line.

In Fig. 2 *A O B* represents the curve of an arch shown in the position in which a parabola is generally considered to be placed with respect to the axes of co-ordinates. For a concentrated load *P*, the triangle *A F B* is an equilibrium polygon. If the distances *D C* and *F E* are equal, this polygon is the one giving the minimum moment on the arch. The amount of this moment is v times the thrust in the polygon. The various relations shown in Fig. 2 may be deduced from the properties of a parabola. The moment on the arch may then be expressed in the following equation:

$$M_1 = \frac{P(S z^2 - 2.4142 z^3)}{S(S - 2 z)} \quad (4)$$

For the position of *P*, to give the maximum value to this moment, we equate the first differential coefficient of this expression to zero. The value of z thus found is .330*S*. A concentrated load would then be placed two-tenths of the span length from one end of the span in order to give the maximum bending moment on the arch ring.



P/2
P/2
Q
F
O
G
D
M
E
C
Thrust = PS/8R ((2S-4.8284Z-Q)/(S-2Z))
B
A
Reaction = P/S (S-2.4142Z-5Q)
2.4142Z
(a)
Load W per. ft.
F
O
G
D
E
C
Thrust = WS/8R ((S-2.4142Z)²/(S-2Z))
B
A
Reaction = W/2S (S-2.4142Z)²
2.4142Z
(b)

FIG. 3.

Substituting the above value of z in (4) we have

$$M_1 = .065PS \quad (5)$$

For two concentrations, as at (*a*) Fig. 3, the equation for the bending moment in the arch ring is

$$M_2 = \frac{P}{2S} \times \frac{(2\,S - 4.8284\,z - Q)z^2}{S - 2\,z} \quad (6)$$

For the maximum value of M_2 we find by differentiating and equating to zero

$$19.31z^3 - 18.49\,Sz^2 + 4S^2z = 2\,Q\,(Sz - z^2)$$

The value of z is seen by this to depend upon that of Q. The following have been worked out:

If $Q = 1/10\,S$, $z = .308S$, $M_2 = .051PS$.
If $Q = 2/10\,S$, $z = .287S$, $M_2 = .040PS$.
If $Q = 3/10\,S$, $z = .267S$, $M_2 = .031PS$.
If $Q = 4/10\,S$, $z = .248S$, $M_2 = .025PS$. (7)

If Q be made zero, we find that $z = .330S$, which agrees with the value for a single load as previously found. If Q be made $= S - 2.4142\,z$, the largest value of Q that would bring both loads on the span, we find that $z = .248\,S$. Hence a greater value of Q need not be considered than this, which is four-tenths of S.

For uniform load, as at (*b*), Fig. 3, the equation for the bending moment in the arch ring is

$$M_3 = \frac{W}{2\,S} \times \frac{(Sz - 2.4142\,z^2)^2}{S - 2\,z} \quad (8)$$

Differentiating this and equating to zero we obtain for the position to give the maximum moment on the arch $z = .248\,S$. Substituting in (8) we have

$$M_3 = .010WS^2 \quad (9)$$

This agrees with the last equation of (7), if P be made $.4\,w$, as it should.

We thus see that the maximum effect of any live load in producing bending is obtained when the load covers four-tenths of the arch.

The foregoing considers only the bending moment on *the arch* produced by the live load, and not the varying *thrust due* to the shifting position of the load. It is there-

fore applicable to the steel reinforcement only, as the steel is assumed to be stressed only by bending on the arch ring and not by direct load or thrust. For the position of live load to give the maximum effect on the concrete we should derive an expression that will combine the extreme fiber stress due to bending and the unit load due to the thrust or direct compression. Taking the differential coefficient of such expression and equating it to zero we would find the position of live load that will give the maximum effect on the concrete. The writer worked out the problem along these lines and found that the resulting equations involved R and D as well as S and Q. However, by assuming ratios between R and D both these terms could be eliminated. Using wide variations in the ratios of R to D it was found that the values of z that satisfied the equations were practically equal to those found by considering the bending moment alone. This is because of the relatively greater effect of the bending moment as compared with the thrust in producing stress. The equations expressing the maximum bending moments on the arch, as already given, will then be used both in discussing the proportions of the arch ring and the reinforcing steel.

This treatment of the reinforced concrete arch demands practically constant reinforcement near both top and bottom surfaces of the arch ring from end to end, with rods running into the abutment for anchorage. The reason for this uniform reinforcement is seen by an inspection of Fig. 4. In this figure the center line of arch is represented by the line $A\ O\ B$. The bending moment due to a single load rolling across the span is represented by the vertical distances from $A\ O\ B$ to the curves $A\ M\ N$ and $Q\ R\ B$. The bending moment due to uniform load advancing across the span is represented by the vertical distances from $A\ O\ B$ to curves $A\ T\ U$ and $Q\ V\ B$. It is here seen that *the bending moment is pretty generally distributed in the length of the arch,* so that no part of the arch can be *lacking in reinforcement.* The bending moment is of o

posite sign for load coming on from the right and from the left, showing the need of reinforcement near the top and bottom of arch ring from end to end of span.

Taking up first the steel reinforcement we will assume that 1¼ per cent of steel area is to be used, ⅝ of 1 per cent near the upper surface and the same amount near the bottom surface of the arch placed ⅛ D from the surface. Referring to the writer's article on beams and slabs, published in *Concrete Engineering,* Jan. 15 and Feb. 1, 1907, it will be seen by equation (1) that for a reinforcement of 1¼ per cent at the bottom of a slab the allowed bending moment for rolling loads is $88 d^2$ per ft. width of slab, d being in inches. For ⅝ of 1 per cent, as here assumed, the bending moment, to give the same stress on the steel, is $44 d^2$. Equating this to the value of the maximum moment from uniform live load, as given in equation (9), and reducing the depth to feet, we have

$$\frac{W S^2}{100} = 6336 D^2 \qquad (10)$$

In the design of arches the following live loads will be taken as standard:

100 lb. per sq. ft. for foot traffic and light vehicles.

200 lb. per sq. ft. for heavy driving.

500 lb. per sq. ft. for trolley traffic.

1,000 lb. per sq. ft. for steam roads.

The latter is the live load found by taking an axle load of 60,000 lb. uniformly distributed over a space 5 by 12 ft. The load of 500 lb. per sq. ft. is probably greater than any live load at present specified for trolley traffic, but the experience of steam railroads teaches that it is wise to anticipate heavier loading. It is especially so in a structure of the permanence of a reinforced concrete arch and one which has so little possibility of being reinforced subsequent to its construction or used again for another location having lighter traffic.

Reverting to equation (10), if we let W = 100, 200, 500, 1,000, *we find*

$80 D$ for light traffic.

$S = 56D$ for heavy traffic.

$S = 36D$ for trolley traffic.

$S = 25D$ for steam roads. (11)

These ratios of span to depth, while not intended for final use in design, are of use in fixing upon a close approximation of the final proportions. The economical proportions can readily be studied by means of equation (10). If more steel be used than 1¼ per cent, a less depth can be employed than that given by equations (11), and if less steel, a greater depth is needed. Steel reinforcement is affected only by the live load, and, as seen, is independent of the rise of the arch, except as the rise is governed by the depth assumed.

To take into account the effect of live load on the concrete we will consider the arch as being loaded for 4-10 of the span, giving the moment as shown in equation (9). By the principles of mechanics, in a rectangle having a width B and depth D

$$M = \frac{KBD^2}{6} = \frac{KAD}{6} \text{ or } KA = 6\frac{M}{D}$$

where M is the bending moment, K is the extreme fiber stress, and A is the area of the rectangle. But $K\ A$ being the product of fiber stress and area may be called an equivalent direct stress, for if we divide this by the area we obtain the fiber stress. Applying this in equation (9), we have for an equivalent direct load L on the arch

$$L = \frac{6\ WS^2}{100D} \qquad (12)$$

Under the same loading, from the expression for thrust as given at (b), Fig. (3), we find it to be

$$T = \frac{4\ WS^2}{100R} \qquad (13)$$

We can now modify the derivation of equation (1) to include the effect of live load on a portion of the span only. The allowed thrust, instead of being taken at 36,000 D to allow for a possible bending moment due to irregularities, will be taken as 72,000 D, as we now have a definite bending moment, which is a large factor in producing

equivalent direct stress as a component part of that thrust. The second term on the right hand side of the equation just above (1) will now be the value in equation (13), and a new term will be added, namely, the value of L in equation (12).

Hence we have, as an equation expressing the proportions necessary in an arch whose central line coincides with the curve of equilibrium for dead load, the maximum effect of live load being considered, the following:

$$72{,}000D=\frac{25S^2}{12}+\frac{4WS^2}{100R}+\frac{25HS^2}{2R}+\frac{165DS^2}{8R}+\frac{6WS^2}{100D} \qquad (14)$$

Both equations (1) and (14) should be made use of in finding the depth of an arch, and the depth that is the greater should be employed. For concentrated loads equation (14) may be modified by altering the second and last terms on the right hand side in accordance with the particular loading used.

If for trial we make $H = D$, using the ratios in equations (11), and solve equations (1) and (14), we find the values shown in Table A. The values in columns 2, 4, 6 and 8 are found by equation (1), and those in columns 3, 5, 7 and 9 are found by equation (14). The approximate agreement between these rises for each class of loading suggests that the percentage of steel reinforcement used, namely, 1¼ per cent of total area, is probably close to the economic and proper amount. As this is not far from the amount of steel used in many arches already built, there is a further suggestion in the table, namely, that the rises shown for the various classes of loading and different spans will probably give close to the right proportions.

It is recommended that reinforcement of an arch be made with round rods running from end to end of span and for a distance of 50 diameters of rod into each abutment. They should be spliced, where they join, with turnbuckles. Rods should lie ⅛ of the depth from both top and bottom surface of arch. Sharp curves should be avoided, *especially in* rods near the surface, for stress in the steel *may cause* the rod to tear out a portion of the concrete.

DIMENSIONS IN FEET.

SPAN.	Live Load 100 Lb. Per Sq. Ft. S=80D.		Live Load 200 Lb. Per Sq. Ft. S=56D.		Live Load 500 Lb. Per Sq. Ft. S=36D.		Live Load 1000 Lb. Per Sq. Ft. S=25D.	
	Rise for Full Load	Rise for Max. Moment.	Rise for Full Load.	Rise for Max. Moment.	Rise for Full Load	Rise for Max. Moment.	Rise for Full Load.	Rise for Max. Moment.
20	1.0	0.6	1.2	0.7	1.7	0.9	2.2	1.0
40	3.2	2.4	3.5	2.4	4.3	2.7	5.2	2.9
60	6.9	5.9	7.0	5.3	8.1	5.7	9.3	5.7
80	12.9	11.7	12.2	9.9	13.1	9.9	14.5	9.6
100	22.3	21.5	19.4	16.6	19.5	15.7	20.9	14.7
125	42.4	43.7	32.3	29.0	30.0	25.6	30.8	23.0
150	81.4	92.2	51.6	48.2	43.8	39.0	43.1	33.6

TABLE B.
ARCHES NOT REINFORCED.

DIMENSIONS IN FEET.

SPAN.	Live Load 100 Lb. Per Sq. Ft. S=65D.		Live Load 200 Lb. Per Sq. Ft. S=42D.		Live Load 500 Lb. Per Sq. Ft. S=26D.		Live Load 1000 Lb. Per Sq. Ft. S=18D.	
	Rise by Eq. (16)	Rise by Eq. (15)	Rise By Eq. (16)	Rise by Eq. (15)	Rise by Eq. (16)	Rise by Eq. (15)	Rise by Eq. (16)	Rise by Eq. (15)
20	1.1	0.8	1.3	1.0	1.7	1.2	2.1	1.5
40	3.7	3.2	3.8	3.8	4.4	4.1	5.2	4.6
60	8.1	7.8	7.7	8.8	8.5	8.8	9.6	9.5
80	15.4	16.0	13.6	6.9	14.0	15.9	15.2	16.4
100	27.0	30.3	21.8	29.4	21.1	25.8	22.2	25.7
125	52.1	65.3	36.3	54.7	32.7	43.1	33.1	41.2

If rods are placed 2 diameters from the surface, the radius of curvature should not be less than about 80 diameters of rod. If rods are three diameters from surface, the radius of curvature should be not less than about 50 diameters of rod. This is to keep the shearing unit on the concrete covering the rod within safe limits.

There should be rods through the arch laid across the main rods and wired to the same, to tie the concrete together in that direction and to distribute the load in a transverse direction. These may have 1-3 to 1-5 the area of the main reinforcing rods.

In the abutment it is necessary to consider (1) the load per sq. ft. on the soil, (2) the stability against overturning, (3) the stability against sliding.

In Fig. 1 the principal forces acting on the abutment are indicated in the parallelogram *A B C D*. In this parallelogram *D C* represents the force supplied by the arch itself. This is the resultant of the total weight of arch, including the live load supported, and the horizontal thrust under full load. The force *B C* is the weight of concrete and earth lying directly above the base *E F* applied in the line of the resultant of these combined weights. *A C* is the resultant of these two. This latter line must pass within the middle third of the base *E F*, so as to insure a condition of zero tension at *F*. The maximum pressure on the base will then not exceed twice the average pressure from the total weight above given, considering this weight as distributed on the base *E F*. To this, however, is to be added a load per sq. ft. equal to the live load per sq. ft. to allow for live load over the abutment.

To provide against sliding, the direction of *A C* should be such that its horizontal projection is not more than one-half of its vertical projection. This means that a coefficient of sliding friction of one-half is assumed.

The dimensions of the abutment are to be adjusted until the above conditions are satisfied. If the resultant pressure does not fall within the middle third of the base, the base must be extended away from the arch. Also if

ıe vertical pressure on the soil is too great, the same ktension should be made; or an offset may be made on ıe inner edge of abutment, if this does not cause the line f pressure to fall without the middle third of the new ıse. If the inclination of the resultant $A\ C$ is not steep ıough to have a tangent of two, the abutment should be ade deeper.

The above does not apply to a rock foundation. Where ıe abutment rests on rock and can be built so as to have bearing against a vertical rock surface to take the hori- ›ntal thrust, it need not have mass enough to resist over- ırning, for the rock surface can be relied upon to resist ›rizontal forces. The inclination of the resultant pres- ıre may, of course, be less, as sliding is also prevented r the rock.

As a corollary to the foregoing it may be added that the ıme general principles apply to stone arches or plain ›ncrete arches, except as they bear on the steel rein- ›rcement. In such an arch, where the material will not ıke tension, the resultant pressure must not fall outside f the middle third of the arch ring.

Fig. 5 shows the conditions that will hold when the :sultant falls at the edge of the middle third of the arch ng. Taking equation (14) we find that the first, third ıd fourth terms of the right hand side of that equation ıual the dead load thrust, and the second term is the live ıad thrust when the live load covers 4-10 of the span 'om either end. The distance $\frac{1}{4}\ R$ in Fig. 5 is the maxi- ıum deviation from the arch ring, or v of Fig. 2, under ıe same loading. Making this substitution in the equa- on in Fig. 5 and reducing we have (if $D = H$)

$$R = \frac{5.521\ D^2 + .0067\ WD}{.01\ W - .3472\ D} \qquad (15)$$

This expresses the relation between the rise and the :pth of an arch needing no steel reinforcement. In such ı arch we could *allow* an extreme fiber stress of 400 lb. *r sq. in., or an* average at the crown of 200 lb. under

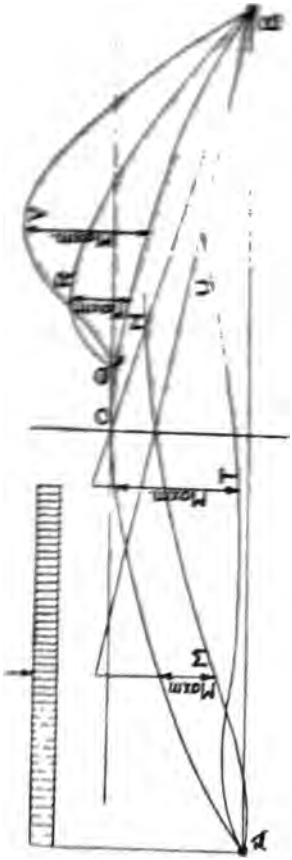

Maxm
Maxm
Maxm
Maxm
A

FIG. 1

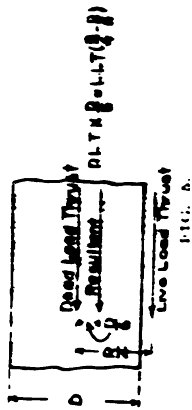

Dead Load Thrust
Resultant
Live Load Thrust
D

FIG. 2

ıll load. Following the derivation of equation (1) we ıay then write

$$28,800D = \frac{25S^2}{12} + \frac{WS^2}{8R} + \frac{25HS^2}{2R} + \frac{165DS^2}{8R} \qquad (16)$$

By trial it was found that using the ratios of S to D iven in Table B the approximate agreement in values of R ere found to exist. It is interesting to note also that ıese values of R do not differ greatly from those in able A.

It is to be observed that the arch of Table A is proporoned on the assumption that the steel reinforcement is :ressed by any eccentric load on the arch ring. In thick rches especially reinforcement could rationally be diminhed by reducing the bending moment by the amount that ıe plain concrete would take. That is, in refining the ılculations the distance v of Fig. 2 can be diminished by .6 of the depth of arch ring.

For short spans of say 20 to 30 ft., arches do not seem › be appropriate. Flat slabs or straight beams and slabs ould seem to be more suitable. Horizontal thrust is then liminated, and the abutments may be much lighter.

SHORT ARCH SPANS.

ditor Concrete Engineering:

Sir: In your issue of March 1, Mr. Godfrey concludes n article on "The Design of Reinforced Concrete Arches," ith the following statement: "For short spans of say 20 › 30 ft., arches do not seem to be appropriate. Flat slabs r straight beams and slabs would seem to be more suitble. Horizontal thrust is then eliminated and the abutıents may be much lighter."

I am unable to discover any basis for such a conclusion. he relations between arches and beams of short spans re the same as for arches and beams of longer spans. If, ıen, a beam of 30 ft. is more desirable than an arch of ıat span, why is not a beam of 150 ft. span more desirble *than an arch of the* same span. The only reasonable *fference between the* two is in forms and erection and

a space is to get an opening for water or passage. With so low a rise the arch would have to be on the top of high abutments in order to get the opening necessary. Again with so low a rise there would be a heavy thrust. High abutments with a heavy thrust on top would be anything but economical.

I attempted to show in the first part of my paper that it is not good engineering to depend on earth as exerting an active horizontal pressure. It is well known that earth settles away from any pressure brought upon it. What sort of conditions would result if a short arch of 20 ft. span settled away at the ends to the extent of only an inch at each end? It is especially true that earth fill is quite unstable. This is what would be encountered in the large majority of cases so near the top of an arch. Such would be totally unfit to take any horizontal thrust.

Mr. Luten says that the relation between arches and beams of short spans are the same as for arches and beams of longer spans. I do not see how he can sustain such an assertion. I do not think that he would attempt to make or approve a beam of 150 ft. of span, and I do not believe he would condemn a flat slab for a span of 5 ft. My table shows a much greater relative and actual rise in arches of long spans. These greater rises not only allow the clearance and opening necessary, but they also reduce the relative thrust and bring the points of its application down near the base of abutment. These are very important differences in the relation between beams and arches in long and short spans.

Answering the latter part of his third paragraph I would say that the "arch" that exerts no horizontal thrust at all is not an arch at all. It is simply a curved beam. A simple truss span may be given any camber, even to a large fraction of the span, and supported on rollers at one end. It is still a simple span and in no sense an arch. So a steel beam may be curved and given only horizontal supports. *It is only a curved beam.* An arch must have thrust to be *an arch.*

There is a popular notion, born of little knowledge, that an arch is the strongest form of construction. But an arch without abutments to take the thrust is not an arch and is not strong. Independent of the effect of curving on the metal a curved I beam is not as strong, simply supported, as a straight one.

EDWARD GODFREY.

CRITICISM OF MR. GODFREY'S ARTICLES ON ARCHES.

Editor Concrete Engineering.

In your issue of April 1, 1907, I called attention to an erroneous statement in Mr. Godfrey's article, that short beams were more efficient than short arches and he replied with the remarkable argument that an arch proportioned for efficiency without considering the abutments, would require such heavy abutments that the beam would be more economical.

Mr. Godfrey continues: "It is a popular notion born of little knowledge that an arch is the strongest form of construction." It is quite true that this notion is born of little knowledge, for the most inexperienced stone mason learns it, but it is surprising that Mr. Godfrey fails to see it. I have been moved to investigate his series of articles on arches and I find a most remarkable combination of unauthorized assumptions, too numerous to be mentioned in one letter. I beg, however, to call your attention to a few on the first page of his article in your Feb. 15 issue. In (4) he states: "In the case of plain concrete arches it is conceded that the arch will be stable *if an equilibrium polygon* can be drawn, using any possible applied loading that will pass within the middle third of all joints." This statement should be *"if the proper equilibrium polygon* for each arrangement of loading can be kept within the middle third." Otherwise it is unsound, and his accompanying statement in which he selects the equilibrium polygon that *gives the* minimum moment, is equally unsound. For an *arch hinged* at the abutments this latter assumption will

hold good only when there is a shortening of the span on striking centers, and every builder of a concrete arch knows that the arch settles and the span increases when the centers are removed. The assumption might prove good if there were great expansion of the concrete in setting, but in all other cases it is not only baseless, but is actually on the side of danger. Many of Mr. Godfrey's conclusions will require modification because of this error.

In (6) and (7) he states that "the office of the abutments as anchoring mediums for the reinforcing rods in the arch will be recognized," and in the next paragraph adds that "the arch will be taken as hinged at the abutments."

In (3) Mr. Godfrey says that "it is not good engineering to depend for stability on earth exerting an active horizontal pressure." It occurs to me that it is good engineering to make the base of a retaining wall sufficiently wide to resist the overturning action of this pressure. Perhaps Mr. Godfrey meant to say that the passive resistance of the earth should not be relied upon, but if so he failed to conform to that idea in his arch design, for he actually does neglect the active horizontal earth pressure. Yet every retaining wall proves that there is such a pressure. It not only must be depended upon for stability, but if neglected it may cause the collapse of the arch. Moreover the passive resistance of earth may be relied on to a limited extent, as Mr. Godfrey himself shows, for his abutment design allows varying intensities of stress.

Many of his assumptions are made to simplify his process regardless of consequences, and he labors to make himself and others believe that they are justifiable.

Mr. Godfrey is wrong in saying that a curved beam is not as strong as a straight beam. The bending moments on the two beams are the same, and the moments of resistance are in favor of the curved beam as an analysis will readily show. If he will compare beams and arches on the basis of *waterway* area he will still find that the arch *is the more efficient.*

He suggests that I would not approve a beam of 150 ft. span. Certainly not, but merely because of the cost. And this is exactly the reason that I urged against the short span beams. A beam of 150 ft. is quite feasible given sufficient depth and material, but it is exceedingly inefficient as compared with an arch. For shorter spans the difference is not so marked, but it is nevertheless in favor of the arch.

I have not by any means uncovered all the erroneous assumptions in Mr. Godfrey's article, and with your permission will conclude in a subsequent issue.

DANIEL B. LUTEN.

MR. GODFREY'S REPLY TO THE FOREGOING.

Editor Concrete Engineering.

I beg to thank you for the opportunity to reply to Mr. Daniel Luten's second letter criticising my paper on arches.

In Mr. Luten's first paragraph he tries to read into my statements an absurdity that does not exist. It is perfectly rational to design a beam entirely apart from the supporting walls or columns. Does Mr. Luten see any absurdity in following this with a statement that this beam requires walls or columns or other support? It is perfectly rational to design an arch without any reference to what is going to support it and to offer the necessary horizontal thrust to make it act as an arch. Is it absurd or "remarkable" then to say that if economy in such an arch demands a low rise, the supporting abutments must be very heavy because the thrust is great?

Now an arch of small span could be made with a large rise in order to get the necessary waterway under it, and this large rise would permit of a small depth at crown because the thrust would be relatively small. So far, so good. Theoretically such an arch would be economical because of the small thickness, *for perfectly balanced static loads.* If such an arch be subjected to live load, because *of the fact* that it is of shallow depth and that it has a *sharp* curve, the equilibrium polygon would readily pass

away beyond the confines of the arch, and the bending moment would be great. An arch of shallow depth would offer but little resistance to bending moments, and the economy of the arch goes aglimmering.

I have heard men express the opinion that camber in a girder or truss span is to prevent the span from dipping from a true horizontal line for fear that the strength will all be gone when the deflection exceeds the camber. Others a little better informed still think that camber is necessary to the strength of a truss span, when in fact it does not affect the strength in any manner, and is simply to prevent a dip in the track. These notions are rudiments of misinformation from the days when structures were designed by judgment, or, in other words, guess. Are we to return to those days and take our knowledge from "inexperienced stone masons," or are we to continue to analyze structures on the basis of principles of mechanics? I would like to repeat my former assertion to give it emphasis. An arch without abutments to take the thrust is not an arch and is not strong.

I would not have Mr. Luten or anyone else imagine for an instant that my treatment of an arch has anything of the exactness of the common treatment of steel structures, or that it is theoretically correct. The first paragraph of my paper sets this forth clearly. Assumptions are absolutely necessary, and a theoretically correct treatment of an ordinary stone or concrete arch is an impossibility. To say that my assumptions are unauthorized leaves me with nothing to reply. One is naturally timid about announcing himself as an authority. If the assumptions are declared to be unsound, that is a different matter. I shall reply from that standpoint.

Mr. Luten's first attack upon my assumptions is rather upon the English with which one of them is clothed than upon the assumption itself. It is instructive to note that it is the proper equilibrium polygon that must be used, that is, not one *belonging* to some other arch that may be *laid out on the board,* but the polygon and loading belonging

to the arch under consideration. As I read literature on arches, the proper equilibrium polygon is arrived at by a system of cut and try until a polygon is found that will fill the bill. Of course this method is based on authorized assumptions, and demands a drawing board and all of the necessary accoutrements to make it a first class guessing match. It would not do at all to ascertain beforehand, by calculation, as could readily be done, where this proper polygon will probably pass.

I cannot see how "each arrangement of loading" is any more lucid than "any possible applied loading." I made no attempt to go into the matter of explaining the well known process of determining the stability of an arch by drawing a polygon that will pass through certain points in the arch, and then, if this fails to pass through the middle third of all joints, drawing another, and another, etc., until one is found that will pass through the middle third throughout. This is known to engineers, and if any others wish to follow that way, they can consult works on arches, where this process is fully set forth and recommended. My treatment of plain concrete arches is nothing more nor less than a systematic method of finding, analytically, how deep an arch should be in order to assure the equilibrium polygon falling within the middle third.

I will not discuss the soundness of this "authorized assumption" until Mr. Luten brings out something more understandable than the last part of his second paragraph. I might state, however, that an arch that is designed to give a horizontal thrust against a vertical wall of yielding earth could not be expected to do anything else but settle and increase in span. This is the very thing I stated in my previous letter as the objection to an arch of low rise and small span. It is objectionable in an arch of any rise or span to depend upon a vertical wall of earth to take the thrust, because the earth will settle away, and the arch, to an unknown extent, becomes a beam.

In Mr. Luten's third paragraph he tries to read another *absurdity* into my paper. It is because a well designed arch

needs reinforcement near the top and bottom from end to end of span that I would run the reinforcing rods into the abutment, otherwise there would be no reinforcement until a point was reached, some distance from the abutment, where the rods had sufficient embedment in the concrete to make them effective. It is because the abutment of an arch would not be suitable anchorage for a cantilever that it would not be suitable to hold the end of an arch fixed. Heavy abutments could be designed that would be suitable for taking the stress of a fixed ended arch, but their economy is very doubtful.

Down in Palos, Alabama, some years ago, I observed some large piers being constructed in earth that was "candy" to dig. These piers not only had vertical sides, but they were chamfered out so as to have a base broader than the section at the ground level. All of this excavation was done without a stick of a shore to keep the earth from caving in. Where was Mr. Luten's "active horizontal earth pressure?" It is not an unusual thing to see trenching done in earth with little or no bracing and to see earth standing in a high vertical wall for some time. The principal forces against a retaining wall are those due to the loosening action of rains or the expanding action of frost. These are wedging actions. They have no place whatever on the haunches of an arch, and they would be broken reeds to rely upon against the side of an abutment. I meant exactly what I said, and not what Mr. Luten thinks I ought to have meant. The horizontal pressure of earth over the top of the arch *may* be neglected with perfect safety. The horizontal pressure of earth behind the abutment *should* be neglected for safety.

Mr. Luten acknowledges that the bending moments of a curved and a straight beam are the same. It would be interesting to see the analysis that shows that putting an I-beam through a bulldozer and curving it in the plane of the web, increases *its* section modulus. The reason why *a curved beam is not* as strong as a straight one is because *the flange stresses,* which tend to go in straight lines, will

in a curved flange, induce secondary stresses. These stresses would tend to increase or decrease the curvature of the outstanding flanges, according as the stress is compression or tension. Curved flanges of a beam or girder are just as irrational and uneconomical as bowed columns or tension members (bowed in one direction.)

Mr. Luten says in this letter that the difference between arches and beams is not so marked in shorter spans. In his former letter he said: "The relation between arches and beams of short spans are the same as for arches and beams of longer spans." Mr. Luten has more confidence in beams of reinforced concrete than I have. As an engineer I would condemn a beam of 150 ft. span. Would Mr. Luten condemn a slab of 6 ft. span?

EDWARD GODFREY.

The Design of Foundations.

The requisites of a good foundation are: (1) The pressure per sq. ft. on the soil must not exceed a certain safe limit. (2) The unit pressure on the entire foundation should be as near uniform as practicable. (3) The pressure should never be negative, that is, there should not be a tendency to lift the foundation which is in excess of its weight at any part. (4) The foundation must be sufficiently deep not to have the underlying soil disturbed. (5) The materials must be practically indestructible in their respective places. (6) The integrity of the foundation itself must be assured; that is, it must be capable of resisting the forces upon it.

To provide for the first requisite the safe bearing power of the soil must be known. This is not determined by experiment so much as by experience.

The pressures allowed by the New York Building Code per sq. ft. on various soils are as follows: Soft clay, one ton; ordinary clay and sand together, in layers, wet and springy, two tons; loam, clay or fine sand, firm and dry, *three tons;* very firm, coarse sand, stiff gravel or hard *clay, four* tons. The same building code allows for tests

being made to determine the bearing capacity in special cases.

In Baker's Masonry Construction the following are given as the safe bearing power of soils in tons per sq. ft.: Quicksand, alluvial soils, etc., 0.5 to 1; sand, clean, dry, 2 to 4; sand, compact and well cemented, 4 to 6; gravel and coarse sand, well cemented, 8 to 10; clay soft, 1 to 2; clay in thick beds, moderately dry, 2 to 4; clay in thick beds, always dry, 4 to 6; rock, from 5 up. This lower value is for rock equal to poor brick masonry.

In the case of hard rock the area of foundation may sometimes be determined by the strength of the foundation rather than that of the rock. Thus, if concrete is used in a pier with a bearing power of 15 tons per sq. ft., this sets the limit, though the rock may be capable of carrying a greater load.

Instability in a foundation, as regards the bearing power of the soil, is exhibited in the sinking or settling of the superstructure. This may be the result either of compressibility of the soil or of lateral flow in it. The unit loads above given are those that will generally give a structure with little or no settlement. On soils other than rock, or solid gravel, or hardpan a little settlement is usually expected and sometimes allowed for in fixing the level of the floors.

Compressible soils may have their bearing power increased (1) by ramming; (2) by driving in short piles to compact the soil by this means; (3) by driving in piles 6 or 8 ft. and then withdrawing them and filling the holes with sand, slag, gravel, or concrete, well rammed in, or the holes may be made by driving a cast iron cone 20 or 30 ft. into the soil and ramming the hole full of the materials named. The latter method was employed in the foundation of some of the buildings of the Paris Exposition, the ramming being done with a cast iron weight of 1 to 1½ tons. (See *Engineering News*, Sept. 27, 1900.)

The *advantages* of monolithic and reinforced concrete *over all other forms* of construction in foundations are seen

in structures resting on yielding soils. The solid mass of concrete, as in a wall, tends to settle as a unit, and uniformly, even though the pressure may not be quite uniform on the entire foundation.

Lateral flow in the subsoil is especially troublesome in soils of a clayey nature or in sand that is saturated with water. Quicksand is a saturated sand that flows very freely, but many saturated sands that would not be classed as quicksands are subject to this lateral flow; and foundations upon such require the utmost care. A good precaution is to drive sheet piling just outside of the foundation, so as to retain the sand or other soil, if flowing is anticipated. This will greatly increase the bearing power. The tower of the Hamburg water works is about 290 ft. high. It is built of brickwork and rests on a circular block of concrete 56 ft. in diameter and 11 ft. thick. This rests on quicksand enclosed by sheet piling driven below the line of saturation of the River Elbe. The pressure is about 2 tons per sq. ft. on the quicksand.

Much trouble is caused in the city of Chicago due to the flowing of the subsoil. In the down-town portions of the city there is first a layer of about 13 ft. of made ground, then 6 to 12 ft. of a hard clay, below which is a softer clay for about 60 or 80 ft. to rock or hardpan. In some places this clay becomes very hard towards the rock and contains large boulders. Before the days of tall buildings foundations in Chicago were generally laid on top of the crust of stiff, blue clay found about 13 feet below the surface. This was loaded with about 1¾ to 2 tons per sq. ft., and in order to get the necessary area in contact spread footings were employed. These are large pyramids that had to rest on top of the hard clay, as it was found that the bearing power was less if they penetrated into the clay. Before building the Masonic Temple the soil was tested by a tank having an area of 2 sq. ft. in contact with the soil. This was loaded until the pressure was 5,650 lb. per sq. ft. In one test it was placed on top of the hard clay for 100 hours; the

settlement was 1 13-16 in. In a second test it was supported at the bottom of a hole 2 ft. 4 in. deep in the hard clay; the settlement was 4⅛ in.

Some of the taller buildings were founded on these spread footings, but it was found that the entire crust was depressed by the great load. The settlement was greater, though the unit pressure was the same as in smaller buildings. This was especially the case where any excavation was made below the crust in the neighborhood of these buildings. Excavation for the freight tunnels, which are about 40 ft. under ground, recently caused some tall buildings to settle and develop bad cracks on account of the flowing of the soil.

The present methods of sinking foundations in Chicago will be described later. Even these cause some settlement in old buildings because of unavoidable spaces left around the lagging in the wells.

Sometimes retaining walls are built around the site of a building below grade. This, however, is to prevent the flowing in of the soil, if excavation is made for a subbasement.

The bearing power of gravel or other similar material may sometimes be greatly increased by the use of grout. Gravel not mixed with sand may readily be consolidated into a sort of concrete by forcing into the interstices cement grout. This has been done also where some sand was present in the gravel, by first pumping out some of the sand. The first method was employed in the foundation of a bridge over the Danube, near Ehingen, Wuerttemberg. Pipes having loosely inserted driving points were driven into the gravel at intervals of 18 or 20 in. By drawing the pipe up a few inches to clear the point grout could be pumped into the gravel. Then by successive withdrawals and pumpings a concrete was made of the natural gravel in place. The gravel was one which contained running water and little sand. This would not work in sand or in gravel where much sand is present. (See Eng. News, Jan. 9, 1902.)

By the same means a sort of cofferdam can be made in gravel by sinking the pipes around the site of the proposed excavation and building up a wall inside of which excavation can be made.

There is a process (patented Dec. 8, 1891) for injecting cement grout into quicksand in the making of a foundation in the same. It consists of (1) sinking pipes into the sand, (2) pumping out the sand so as to leave a chamber, (3) forcing cement grout into this chamber. (See *Eng. News*, April 28, 1892; June 28, 1894; Oct. 16, 1902.) One way to get out the sand is to use two pipes near together, forcing water into one while the water and sand are being pumped out of the other.

In the foundation work of the Merrimac River bridge at Newburyport, Mass., cylindrical piers were sunk by open dredging to a level where rock was indicated by the borings, and the bottom was found to consist of boulders in a stratum of coarse sand and gravel. The cylinders were then converted into pneumatic caissons, with air locks placed on top, and were loaded with pig iron. The water in them was then blown out. No rock ledge was found within reach of steel probing rods 12 ft. long, and it was decided to use the boulders and gravel as the foundation. A foot or more of water was allowed to rise in the cylinder, and Portland cement was mixed with it and kept well stirred. By increasing the air pressure this grout was forced out into the surrounding gravel and boulders. The mass was thus rendered stronger to receive the foundation. (See *Engineering Record*, Vol. 50, page 220.)

When soil is deemed too soft to support the weight of a structure, piles are sometimes driven in. These support the weight either by virtue of their penetrating to hard bottom or by friction on the surrounding soil. Where a substratum of rock can be reached, the piles should be driven to the same, and driving should cease as soon as this is reached. Further hammering may broom or split the pile or cause it to fail by diagonal shear and thus destroy its usefulness. There are various rules for de-

termining when sufficient hammering has been done. One rule allows a penetration not over 16 in. in the last ten blows of a 2,600-lb. hammer falling 15 ft. Another rule allows a penetration not over 1 in. in the last blow with a 2,000-lb. hammer falling 10 ft. Another allows a penetration not over 3 in. under a 2,000-lb. hammer falling 15 ft. for piles supported by friction only. One rule for piles driven with a steam hammer, the hammer of which weighs 5,000 lb., requires that the piles will not move more than ¼ in. to the blow, when driving is complete.

By the New York Building Code piles intended to sustain a wall, pier, or post must be spaced not more than 36 or less than 29 in. in centers. They must be driven to a solid bearing if practicable to do so. Piles less than 20 ft. in length may be 5 in. at the small end and 10 in. at the butt. Piles more than 20 ft. in length must not be less than 12 in. at the butt. The maximum load allowed per pile is 20 tons.

A rule quite general in Boston is to allow a safe load of 10 tons per pile when supported by friction. Piles reaching hard stratum may be loaded to 16 tons. (See *Eng. News,* February 5, 1903).

Specifications for bridges in the city of Milwaukee allow a load of 12 tons on each pile. Piles must be not less than 14 in. at butt and 9 in. at small end and 55 ft. long, driven to hard bottom. The minimum spacing is 2 ft. on centers. Piles may be of pine, tamarack, or hemlock.

The "Engineering News Formula" for the safe load on piles allows a load in tons equal to twice the weight of hammer in tons times fall in feet divided by one plus penetration of pile under last blow, in inches.

To prevent the splitting of the head of a pile a steel ring about 3 in. wide and ½ in. or more thick and about the diameter of the head is laid on it and driven into the fibers with the hammer. This is pulled off by means of a tool for the purpose when the driving is completed. Where *one piece of timber* will not reach to the required depth, it *is sawed off and* spliced by means of a dowel and sid

straps of wood or iron. Piles are usually sharpened with a blunt point. Sometimes steel straps are laid across the point two ways and spiked on, and sometimes cast iron shoes are used to facilitate the penetration and to protect the pile.

Timber piles in permanent structures should only be used where always wet. The piles are usually sawed off at an even level, below low water line, and the earth is excavated around them for 2 ft. or more. This is then filled with concrete and the pier footing of concrete laid upon the same. Care must be taken in laying masonry upon the concrete to see that it is set first.

Cheap woods, such as spruce and hemlock, are generally used for piles completely under water, as they will last indefinitely if kept from contact with the air. These soft woods can be obtained in lengths from 20 to 50 ft. and diameters 10 to 12 in. at the head. Hard pine piles can be obtained up to 75 or 80 ft. in length with diameters at the head of about 16 in.

Concrete piles have recently come into extended use. These may be made by filling up with concrete the hole left by a pile of wood or metal. In some a sheet metal shell covering a removable wooden core is driven into the ground and then the core withdrawn and the shell filled with concrete. These are often tapered. They are generally of larger diameter than wooden piles.

The following method was employed in some concrete pile work at Pittsburg in 1904. Steel tubes about 16 in. in diameter, of metal about ⅛ in. thick, having bullet shaped shoes loosely inserted in the ends and with wooden heads, were driven into the ground by means of pile drivers. Where the ground was hard, these shoes were of cast iron; and where sufficiently soft, they were of concrete previously made and allowed to harden. When driven to the required depth, the tube was drawn up a little; and a rammer, made of long cast iron weight suspended on a rope, was let into the inside. The known length of the line supporting this rammer served to in-

dicate whether the shoe was in the proper place (not being drawn up by the tube); or, if a concrete shoe had been used, it would show whether it had remained whole and had not been jammed up into the tube. Concrete buckets made of old house boilers were then filled and the concrete deposited into the tubes through gates in the bottom of these buckets. After each bucketful had been deposited and rammed the tube was drawn up a little further, the distance being gauged by a mark on the line suspending the rammer. The buckets were not let into the tube, but were emptied at the upper end of the same. Parts of this process are patented.

Piles reinforced with steel are sometimes molded at the site, and after setting and hardening are driven in the same manner as wooden piles. A method of protecting the head of such a pile in driving is described in *Eng. News,* December 27, 1906. It consisted in a steel tube fitting over the top of the pile and having a diaphragm against which on one side was a long wooden block to receive the blows and on the other side a short wooden block, and between this and the head of the pile a cushion of rope.

The foundations of tall buildings in Chicago are now generally made on what might be called concrete piles. They vary from 3 to 12 ft. in diameter and are sometimes 100 ft. long or more, reaching down to hardpan or solid rock. The excavation is done by hand in depths of 4 or 5 ft. at a time. This circular hole is sheathed with vertical lagging made of boards 2 or 3 in. thick planed radially on the edges and fitted tightly together. These are held in place by flat steel bands, segmental in shape and flanged on the ends, bolted together to form a complete circle. Then the excavation is made another 4 or 5 ft., and this is also surrounded by lagging. At the bottom, if the pile is to rest on hardpan, the well is belled out to twice the diameter. The piles are generally loaded to about 20 tons per sq. ft., and this would give a load of 5 tons per sq. ft. on the hardpan. The holes or wells are filled with concrete, well tamped, which should preferably be let down

in buckets, so as not to separate the ingredients and thus impair the uniformity of the concrete. Sometimes the lagging is left in place, and sometimes it is removed as the concreting progresses. The concrete is best made of a mixture of 1 Portland cement to 2 sand to 4 broken stone or gravel. Sometimes 1:3:5 concrete is used. Where a pocket of quicksand is encountered, it is apt to flow into the well and to make a change in the method of excavation necessary. One method of meeting the difficulty is to use a steel cylinder, in segments bolted together, either allowing it to sink as the excavation progresses or forcing it down with jacks. This is not always successful, and resort must be had to the use of sheet piling driven from the surface through the stratum of quicksand. Thus a sort of cofferdam is made around the well inside of which excavation can be done without difficulty. When the sand is penetrated, the first described method, using short lagging, is resumed. Steel sheet piling is best for the purpose, as it will penetrate further wthout injury, and it can be drawn and used again.

The tops of these concrete shafts are capped with grillage beams or other means of distributing the load of the column uniformly over the concrete.

The load allowed on concrete piles should not exceed 20 tons per sq. ft. for those of large diameter. If there is any possibility of their acting as columns, as in the event of the surrounding earth being removed, the unit load should be less. Concrete is weak in columns, unless it is properly reinforced with steel. A better load on piles of small diameter is about 15 tons per sq. ft.

Screw piles are sometimes made use of to distribute pressure and to anchor structures such as lighthouses, signal towers, etc. They are made of a shaft of steel or cast iron and an auger shaped blade of about one turn. They are driven in by turning either by hand or other power. Screw piles for supporting parts of the tunel for the Pennsylvania and Long Island R. R. under the Hudson River are to be made of cast iron in 7 ft. sections, having inside

flanges at splice which take 4 1⅜ in. bolts and 12 steel dowels. The piles are 27 in. outside diameter and of 1¼ in. metal, and the diameter of the screw is 4 ft. 8 in. They are spaced 15 ft. and aid in the support of a single track tunnel. (See *Eng. News*, Oct. 12, 1903.)

The load coming upon the soil of a foundation or on a system of piles consists of the total dead load carried and some or all of the nominal live or superimposed load that the structure may be called upon to carry. In tall buildings it is not necessary to include all of the live load, as the floors are never all fully loaded with the calculated capacity. The New York Building Code allows a reduction in column loads of 5 per cent of the live load of 2 floors (including the roof); 10 per cent of the live load of 3 floors; 15 per cent of 4 floors; down to 50 per cent of 11 floors, and 50 per cent reduction for any more than 11 floors.

In estimating the weight on a foundation part of which is permanently under water it is legitimate to deduct the buoyancy of the water for the part of the foundation that is below low water line. Thus in piers at Chicago and Milwaukee 62.5 lbs. per cu. ft. of masonry below water level may be deducted from the weight of piers. On the other hand this same deduction should be made in determining the stability of a partly submerged pier used as an anchor against uplift or horizontal forces.

A brief description of the processes used in excavating for foundations will be in place here. The ordinary process of excavating for foundations where water is not encountered is simple and the problems are few. Apart from digging or blasting out the material and handling the same there is often the question of shoring up the sides against a cave-in. In a wide excavation in loose ground the shores should not be merely horizontal struts, but these struts should be braced together diagonally and vertically *to prevent displacement*. Wedges or jacks should be used *to give a firm bearing* against the earth, as it is muc

easier to prevent earth from sliding than to stop it when it has once loosened and started to slide.

There is a process of excavating through flowing soils known as the freezing process. It is expensive and not used very much. It consists of forcing into the soil just outside of the opening to be made refrigerating pipes freezing the mass, excavating and then damming off the soil or building in the stone or concrete work.

There are three methods of excavating for foundations in water. One is by making a cofferdam by driving sheet piling around the space to be excavated and digging out the earth. The water is kept pumped out as the excavation proceeds. Wooden sheet piling, called Wakefield piling, consists of boards spiked and bolted together in threes, the middle one being set back to form a tongue at one side and a groove at the other. Steel sheet piling has been found to be very useful for cofferdam work. It has greater strength than wooden piling, and there is less leakage. The piles can be used repeatedly. Sometimes they rust together in the ground. A blow from the pile driver on the head of one pile will generally loosen the one to be drawn.

A second method is called open dredging. This consists in dredging out the earth in the inside of a casing, which sinks as the earth is removed. The casing forms a shell for the pier, being filled with concrete when sunk to the desired depth. The shell has a uniform outside diameter and is tapered from the inside to a cutting edge. The dredging is done by means of steam shovels, or clam shell, orange peel or other bucket dredges.

Hydraulic dredging, used in different methods of excavation, is done by means of pumps. Where loose materials are to be removed by pumping out, a jet of water agitating the materials will cause them to be drawn up by the pump. Jets of water may be used to advantage in *open dredging* to loosen the soil under the cutting edge. *Sometimes* the cutting edge encounters boulders which

the dredge will not remove, and it is necessary to send down divers to remove the same.

By use of the open dredging process in the foundation of the Atchafalaya Bridge at Morgan City, La., the depth of foundation bed was made about 120 ft. below high water and 70 to 115 ft. below the silt or mud surface. The greatest depth attained by this method in a bridge foundation was reached in sinking the piers of the Hawkesbury Bridge in Australia, namely, about 170 ft. below water or 126 ft. below the river bed.

Concrete deposited in deep water, as in an excavation made by open dredging, is apt to have the cement washed out. To overcome this it may be dropped through a tube or a tremie in as large loads as practicable. If put into jute bags, the cement will be retained; enough cement will ooze out of the meshes to cement the pieces together. Concrete mixed extra long or even retempered concrete, if it has not stood too long, is preferable to concrete in which the cement is too freshly mixed, where it is to be deposited in water.

The other method of excavation is the pneumatic process. An airtight timber crib or caisson is made, having a space underneath large enough for men to work in, provided with a cutting edge around the periphery and supplied with air locks, etc., in the roof. This is placed in the position which the pier is to occupy and allowed to rest upon the ground. Men enter and leave through the air locks, and the excavated earth is hauled up in buckets through locks for the purpose. Air is continuously pumped in and it escapes below the cutting edge. Ordinarily this air pressure keeps the water out, but if the soil becomes dense or is clayey, the air pressure can often be reduced below the hydraulic head of the cutting edge, greatly to the benefit of the workmen. In such case the water that leaks in may be removed with an ejector.

As the crib sinks the pier is built on it, and when suitable bottom is reached the working chamber is filled with concrete.

In the Brooklyn pier of the new East River Bridge a depth of 115 ft. below high water was reached by the pneumatic process. In a bridge over the Barrow River in Ireland a depth of 127 ft. below high water was reached. In this work the air pressure used was 42 to 45 lb. at the maximum depth. (See *Eng. News,* Vol. 53, p. 607).

In foundations for tall buildings in New York sometimes pneumatic caissons are required. These are sunk under the individual columns or under two or more columns in a group. (See *Eng. Record,* Sept. 3, 1904, for description of caissons under Trinity Building.)

The second requisite of a good foundation, namely, a uniform unit pressure on the entire foundation, has special force in foundations on soft soil. On such soils there will be some settlement, and if the unit pressure is greater at one point than another, settlement will be greater at that point. This condition of uniform pressure is effected by making the area in bearing on the soil in proportion to the load to be carried. A great part of the settlement of a building takes place during erection, and it is hence due to the dead weight in greater measure than to the superimposed weight. In any event, as only a part of the superimposed load is on the floors continuously, the dead load must be taken as the prime factor in determining the size of footings. If the footings of all of the columns of a building were given areas in proportion to the total dead and live load to be carried, those carrying the walls would settle more than the interior columns. However, the areas of the footings for the interior columns should be based on the probable maximum load, so that the safe pressure on the soil is not exceeded.

Sometimes the density of the soil varies under different parts of a building. In such case the foundation should be proportioned so that the pressure will be in conformity to the bearing power of the soil at different parts. The leaning tower of Pisa seems to have acquired the lean which has made it famous by uneven settling on soils of different densities.

Eccentrically loaded piers resting on piles should have the center of gravity of the system of piles coinciding with that of the load as near as practicable.

The third requisite, namely the maintenance of a positive pressure on the soil at all parts, has special force as applied to foundations for high or narrow structures where the wind may cause tension or uplift on the windward side at the edge of foundation, also for anchorages of cantilever or suspension bridges.

In order to have no tension on the extreme edge of a rectangle, the resultant of the vertical load and the horizontal force (as the wind load or pull of anchorage) must fall within the middle third of the base. In a circular section the resultant should fall within the middle quarter of the base. This is a condition which follows upon the theory of flexure. It does not mean that there is a factor of safety of three when the resultant falls within the middle third of the base. It simply means that this condition must hold, if there is to be no tension and no tendency to rise, at any part of the base. In a material incapable of taking any tension, or nearly so, such as a wall resting on the soil, or a wall laid in lime mortar, there should be absolutely no tension or tendency to rise anywhere. Such tendency would be detrimental to the safety of the structure, especially if it be a reversible condition, as, for example, in a tower where joints may tend to open on one side and then on the other, due to reversal of the wind. Of course it is true that one application of the wind load would not overturn the tower, if the resultant fell somewhere between the middle third and the outer edge, as the resultant must fall without the base in order to overturn a body. But the racking will loosen the joints and tend ultimately to ruin the walls. In a foundation it is manifest that any such disturbance in the region of the main supporting medium of a structure would be harmful.

The condition requiring that the resultant fall within *the middle third* of the base may be seen from another *standpoint, namely,* that of uniformly varying pressures

Suppose the resultant pressure falls in the middle of the base of a foundation as at (*a*), Fig. 1. The reaction of the soil will be uniform across the width of the base, as represented by the row of short arrows. If the resultant of the pressure falls 1-3 of the base from the edge of the same as at (*b*), the reaction will be greatest in intensity at the edge nearest this resultant, and it is necessary that the center of gravity of this reaction be on the line of the resultant. Again, if the reacting medium be yielding or elastic, the pressure will vary uniformly from the maximum at the right edge of the base to a minimum at the left edge. Only a triangle, as indicated in the figure, can fulfill these conditions. Hence the pressure varies from zero at the left edge to an intensity double the mean pressure at the right edge.

If the resultant is closer to one edge than 1-3 of the base, and the variation of pressure is uniform, one of two things must occur; either there will be tension on the farther edge, or the pressure will be zero for whatever width of base remains in excess of $3x$ [See Fig. 1, at (*c*)]. A floating rectangular block supporting an eccentric load whose resultant, or center of gravity, falls closer than 1-3 of the width from the edge would be out of water for the width of block exceeding $3x$.

Fig. 2 shows the anchorage of a suspension bridge. The force $A\ B$ is the pull on the cable; $A\ C$ represents the weight of the anchor pier, applied at the center of gravity of the pier; $A\ D$ is the resultant. This latter force should pass within the middle third of the base

Fig. 3 shows the pier for a trunnion bridge, such as the bascule bridges of which a number are found in the city of Milwaukee. The force A is the total dead load of trunnion and approach girders. This will be at the center of column supporting these loads, as the trunnion girders are counterbalanced for dead load. The force B is the dead weight of the pier, allowance being made for the buoyancy of the water. C is the sum of the live load overhanging the pier. The resultant of all of these is in amount

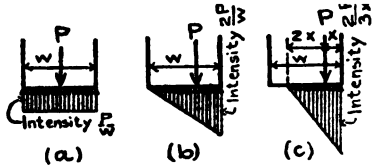


FIG. I.

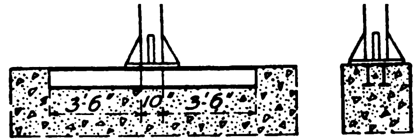
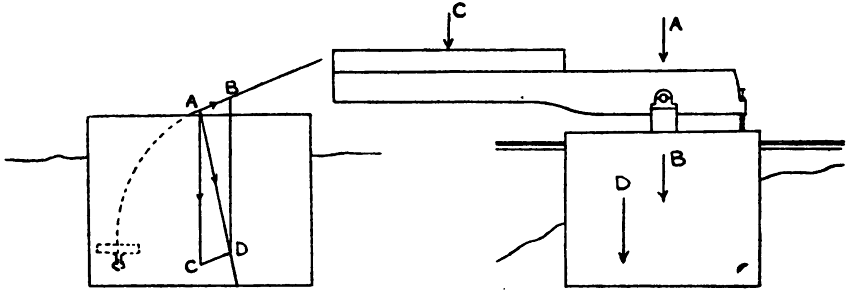


FIG. 4.



FIGS. 2 AND 3.

equal to the sum of the three. Its position is found by taking moments, say around the edge of the pier. It is preferable that this resultant fall within the middle third of the base, but if the pier is founded on piles, they may be driven closer on the side of the higher pressure, thus allowing the resultant to fall closer to that edge.

In the foundation of a chimney or tower the same principles apply. The dead weight of the structure is one force always present, and the greatest possible wind load is to be combined with the same. The direction of this resultant must be such as to pass within the middle third in the case of a rectangular base or within the middle quarter in the case of a circular base, that is, it must fall not over 1-6 or 1-8 of the diameter of the bases respectively, from the center.

It is legitimate to allow somewhat greater pressure at the edge of a foundation than those given heretofore, where the increase is due to the maximum wind load. An increase of 25 per cent over the unit regularly allowed would be a reasonable allowance at the edge where the pressure is of maximum intensity.

The fourth requisite would demand that foundations be made deep enough to be free from danger of undermining by abrasion from streams or drainage water, or by excavation for foundations of adjacent structures. They should be deep enough to rest on soil not affected by frost.

Many failures of bridges have been due to the washing away of the soil beneath the piers. Gravel beds, upon which piers often rest, could very often be cemented to advantage into one mass by the use of grout, as hereinbefore described. Often the scouring action of the stream will carry away large stones of the piers themselves. These stones lose nearly half their weight when submerged, and are hence comparatively easy to move. This is a strong argument for solid concrete piers.

A depth of 4 or 5 ft. is sufficient to reach soil not *affected by* frost in temperate regions. This is deep enough

for light foundations as for mill buildings, etc., where the soil is not made ground or fill.

The fifth requisite of a good foundation, namely, that the materials be practically indestructible in their respective places, can be assured only by using materials of known lasting qualities. Brick should not be used in sea water. Wood should be used only where it will be always under water or always exposed to air only. Cement mortar and not lime mortar should be used in wet places, as lime mortar requires a long time to harden if kept wet. Steel placed in concrete is probably better not painted, as the concrete will adhere better to the steel than to the paint and is a better medium of protection than paint.

The sixth requisite demands a foundation that is strong enough to do the work that it may be called upon to do. The forces to be resisted may be (1) a downward force due to the weight of the structure carried, (2) an upward force due to an uplift that may be exerted upon the foundation, (3) horizontal or overturning forces, (4) the upward reaction of the supporting soil.

The base of a steel or cast iron column or a bridge bolster or shoe resting on stone or other masonry should have sufficient area in contact with the stone to prevent crushing. It should be borne in mind that generally such bases do not have an ideal bearing, so that the unit employed should be low, that is, a large factor of safety should be used It is true that building columns and some other bases are often set up a little above the surface of the stone or concrete and the space filled in with grout, but it is also true that bridge seats are very often placed directly on the surface of the stone. The following are good units of safe pressure to allow on various classes of masonry, in lb. per sq. in.: Brick masonry in lime mortar, 150; brick masonry in cement mortar, 200; ordinary rubble masonry, 200; good bridge masonry, 250; granite, 400.

It is of great importance that the masonry pier receiving a *cast column base* be a rigid mass under the base. The writer observed a number of cast column bases on rubble

piers that were split in two by the weight of the column. The rubble appeared to sink under the center of the column leaving the cast base supported on its edges. Not being suited to take such a heavy bending moment the cast base broke under the weight. The cap of a pedestal or pier taking a column should be a single stone, or, better, the entire pier should be a monolith of concrete.

A large building in Pittsburg collapsed some years ago on account of the fact that a brick wall between two buildings that were being thrown into one, was taken out for one story and replaced by columns about 16 ft. apart. These columns were supported on the rubble wall, which, being intended only to carry a continuous brick wall, was unfit to carry the concentrated weight of the columns. Spreading beams under these columns would doubtless have saved the building.

Suppose in Fig. 4 the load on the column is 150,000 lb. At 200 lb. per sq. in. on the rubble wall a flange area of beams of 750 sq. in. would be required. Assuming I-beam flanges 4 in. wide a length of 188 in. or 15 ft. 8 in. is needed. This will be made up of 2 beams 7 ft. 10 in. long. The pressure per lineal foot is $200 \times 4 \times 12 = 9{,}600$ lb., and the span of the cantilever is 3 ft. 6 in. The bending moment is 705,600 in. lb. and the section modulus required at 16,000 lb. per sq. in. is $705{,}600 \div 16{,}000 = 44.1$. The size of I-beam required is then a 12 in. 40 lb. beam. As the flange width is more than 4 in., a revision of the calculations could be made using the 5¼ in. width of this beam and trying again for the bending moment.

Beams and rails are often imbedded in concrete footings and pedestals to distribute the pressure over a greater surface, the beams being used to give the resistance to bending which the concrete lacks. Rails are not economical for this purpose, unless they happen to be old rails on hand or purchasable at a much lower rate per lb. than I-beams. For the same weight of metal much greater rigidity can be obtained in light weight I-beams than in rails. The concrete, instead of being counted upon to assist the steel

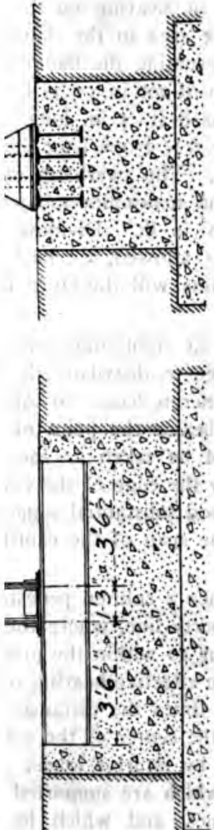


FIG. 5.

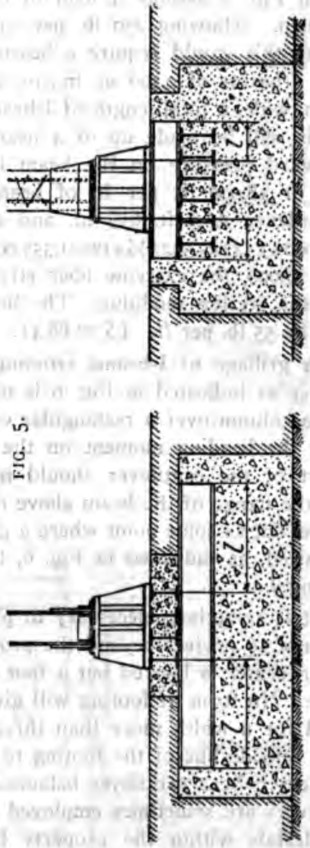


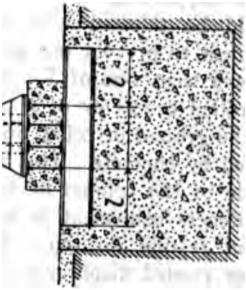
FIG. 6.

is merely the medium to transmit the upward pressure of the soil into the beams, to hold them against buckling, and to protect them from rust.

In Fig. 5 assume a load of 600,000 lb. on the column shown. Allowing 250 lb. per sq. in. bearing on the concrete this would require a bearing area in the flanges of the I-beams of 2,400 sq. in., or, assuming the flange to be 6 in. wide, a total length of I-beam of 400 in. or 33 ft. 4 in. This will be made up of 4 beams 8 ft. 4 in. long. The upward pressure on the beam is $6 \times 250 = 1{,}500$ lb. per in. or 18,000 lb. per ft. of beam. The overhang of the cantilever is 3 ft. 6½ in. and the maximum moment is $18{,}000 \times 3.542 \times 3.542 \times \frac{1}{2} \times 12 = 1{,}355{,}000$ in.-lb. Dividing this by 16,000, the extreme fiber stress allowed, we have 84.7 as the section modulus. The beams will therefore be 18 in. I's 55 lb. per ft. ($S = 88.4$).

A grillage of I-beams crossing at right angles to each other as indicated in Fig. 6 is used to distribute the load of a column over a rectangular concrete base. In calculating the bending moment on the beams the point of support of the cantilever should not be taken as the edge of the flange of the beam above or the edge of the column base, but as some point where a good substantial support is assured, as indicated in Fig. 6, the span of the cantilever being l.

It is sometimes necessary to place a column pedestal so as not to project beyond the property line, where the column center is located but a foot or so within the property line. No form of footing will give effective bearing on the soil for a width more than three times the distance from the outside line of the footing to the center of the column, unless it be a cantilever balanced by interior loads. Cantilevers are sometimes employed which are supported on 2 pedestals within the property lines, and which in turn support 2 columns. Figs. 7 and 8 illustrate cantilever supports of wall columns both for a narrow building, where the 2 outside columns may rest on the same set of cantilever beams, and a building where an interior column



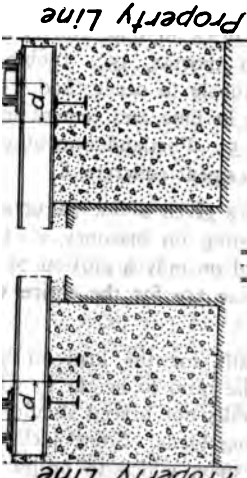


FIG 7.

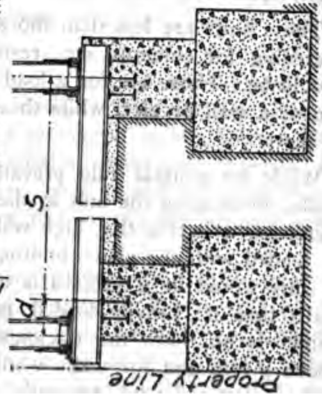


FIG. 8.

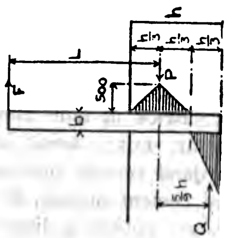


FIG 9.

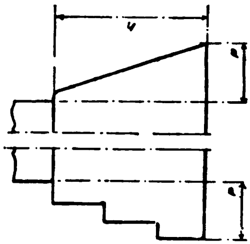


FIG. 10.

is utilized to take the uplift of the cantilever. The span of the cantilever beams is the distance from center of column to the center of the group of supporting beams. The depth of cantilever beam should be selected with the length S in view, making it sufficient to prevent too great deflection upward, say not shallower than 1-20 of S. The cantilever may be composed of rolled beams or of a box girder. If below the surface of the ground or cellar floor, the steel work should be protected by concrete. The box girder may be riveted to the side of the column or may have interior diaphragm and stiffeners equivalent in area to the column, with the column resting on top. The I-beams should have separators or riveted diaphragms between them.

The safe carrying power of masonry, according to the New York Building Code, is as follows, in tons per sq. ft.: Brick in lime mortar, 8; brick in lime and cement mortar, 11.5; brick in cement mortar, 15; rubble in Portland cement mortar, 10; rubble in other than Portland cement mortar, 8; rubble in lime and cement mortar, 7; rubble in lime mortar, 5; Portland cement concrete, 15; other than Portland cement concrete, 8.

These units are less than those given a few paragraphs preceding for columns, etc., resting on masonry, as they should be. Those are for a load on only a portion of the top of a wall or pier, while these are for the entire wall or pier.

While no general rule prevails for the load on high walls, diminishing the unit as the ratio of height to width increases, it is true that high walls not braced should not be loaded with the above limiting loads. Some such rule as this would be a safe basis upon which to proportion walls that are heavily loaded, namely; for walls whose height is 10 times the thickness, or less, use the units above given, and for walls whose height is 25 times the *thickness*, use 2-3 of the same, walls between 10 and 25 *times the* thickness to have proportionate units between

these limits. Walls more than 25 times their thickness in unsupported height are to be avoided.

For an upward pull, such as the anchorage of a cantilever or the uplift of the post of a railroad bent due to wind load, rods or bars should go down deep enough into the masonry to take a weight of the same 50 per cent in excess of the calculated uplift. If the masonry is brick or rubble, there should be a grillage of beams or rails so arranged that they would lift the necessary weight of masonry. For very heavy anchorages in concrete, grillages should also be provided. For a small uplift in concrete a good sized washer plate is usually sufficient. The anchor bolts should have a square head, and the washer plate should have a small stop plate riveted on it to keep this head from turning. Cast washers may be made with a recess in which the head of the bolt will fit. A split bolt and wedge should not, in general, be depended upon for anchorage against an uplift. The holding power of such a bolt is more or less uncertain, and the chances are many that the bolts will not all be properly set.

These split bolts, passing into the masonry a foot or two, are commonly used for anchorage of stringers and girders and trusses of bridges where no uplift is counted upon.

Columns for office buildings are not usually anchored. The cast steel or cast iron bases are generally left untooled on the bottom, and holes are cored in the bottom plate through which grout is poured after the base is leveled up.

No anchor bolts of any kind should be close to the edge of a concrete wall or pier, as the driving may break out the concrete

The stability of a masonry pier against horiontal or overturning forces is met largely by the weight of the pier. Figs. 2 and 3 illustrate piers subject to such forces, and the remarks made in regard to these figures relate to provision for their stability, treating the vertical pressure *on the soil only.* In addition, if the forces are horizontal *or have horizontal* components, there is provision

against sliding to be considered. If we assume a coefficient of friction of the pier on the soil as ½, we should have a horizontal projection ½ of the vertical projection in the line of the resultant pressure. Any earth packed against the side of the pier will be an additional safeguard against sliding. It is best in the case of a constantly exerted force not to depend upon earth exerting horizontal pressure against a vertical surface, as the force will tend to compact the earth and move the pier.

In the case of a pier for a mill building column or a crane runway column the horizontal forces due to the wind against the building or due to the thrust of the crane are only occasionally exerted and very seldom to their full extent. It is safe in such cases to allow for some horizontal pressure exerted by the earth.

To arrive at a means of finding approximately the stability imparted by the lateral pressure of the earth, take the case shown in Fig. 9. This represents a pole placed in the ground and subject to a lateral force. The weight here is negligible If we assume that the pole turns about a point 2-3 of the depth below the ground, the resistance of the earth will vary uniformly from this point down, and would vary uniformly from the same point up, but for the fact that this would make the maximum pressure occurring at the ground level. Because the lateral pressure in ordinary earth at the ground surface would be practically nil, it is taken as varying from zero at the ground to a maximum somewhere below the surface. This is arbitrarily taken at 1-3 of the depth from the surface. It is safe to say that well compacted soil will stand a lateral force, for a short time, a few feet below the surface, of 500 lb. per sq. ft. and about twice this amount at a depth of 5 or 6 ft. below the surface, all without appreciable settlement. Now if P be assumed as constant, Q will depend in amount upon F, as it must be the difference between P and F. For a small force at F and a long lever arm L, Q will approach equality with P. Hence the

approximate moment of stability in ft.-lb. about a section at the force P will be

$$M=500\times\frac{h}{3}\times\frac{5}{9}\times h\times b=92h^2b$$

Thus a pole 1 ft. in diameter, 30 ft. above the ground and 6 ft. in the ground would stand 3,350 ft.-lb. or a force of 105 lb. horizontally at the top. If the ground is hard and rocky or if there is a pavement around the pole, it will stand much more than this.

The moment of stability of a column and pier due to their weight is equal to the product of the total weight (under the condition of maximum horizontal pressure) and 1-6 of the base.

To take an example, suppose a column supporting a crane runway is anchored to a pier 6 ft. deep and 5 ft. by 4 ft. in plan. The pier on account of its being planted in the ground will have a moment of stability 2 ft. below the ground line of $92\times 36\times 4 = 13{,}250$ ft.-lb. On account of its weight (taking the weight of the pier as 18,000 lb. and the minimum load on the column under the condition considered as 10,000 lb) a moment of stability of $28{,}000\times 1=28{,}000$ ft.-lb. at the base of pier. At a height of 20 ft. above the ground the former moment would allow a force of $13{,}250 \div 22 = 602$ lb. and the latter would allow $28{,}000 \div 26 = 1{,}080$ lb. A total thrust of 1,682 lb. could be exerted at this height. In general 2 columns of a runway will be acted upon by the sudden stopping of a load carried by the crane. The coefficient of sliding friction is usually taken as 1-5. The load on the trolley could then be $2\times 5\times 1{,}682 = 16{,}820$ lb. This pier would then do for about an 8 ton crane runway. In a similar manner the stability of a mill building column against wind loads may be worked out. No rigid analysis has been attempted of the forces in Fig. 9, but simply a rough approximation of the stability imparted to a post or pier planted in earth which is tamped around it.

The *allowed slope on* a concrete footing, such as either *of the two forms* shown in Fig. 10, may be investigated

as follows: The bending moment under the edge of wall, at S tons per sq. ft. upward pressure of soil is

$$\frac{2000S}{144} \times \frac{P^2}{2}$$

The resisting moment, at 40 lb. per sq. in. safe modulus of transverse strength is $40\ h^2 \div 6$, both on a rectangle h in. deep and 1 in. wide. Equating these we find the following to be very nearly true:

$$Sp^2 = h^2$$

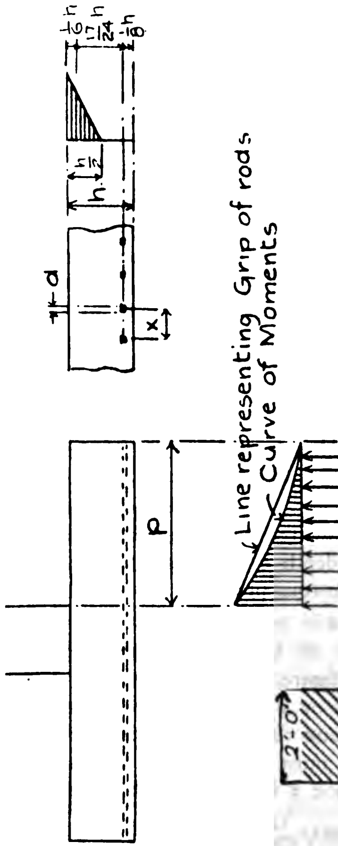
Thus for a pressure on the soil of 2 tons per ft. the depth of the footings should be 1.4 times the projection beyond the wall. For a 4 ton earth pressure the depth should be 2 times the projection.

In a brick footing no reliance can be placed upon tensile strength of the mortar in the vertical joints, but independent of this there is a tensile strength in a brick wall derived from the bonding of the bricks. The friction of each brick against the upper and lower adjacent tiers resists the tendency to draw it out of place. Assume the coefficient of friction to be ½. If the earth pressure at the base of the footing is 1 ton per sq. ft. there will be 1-9 of this pressure on the top and on the bottom of the half of a brick. Hence 1-9 of 2,000 lb. will be required to pull out a brick. This on 16 sq. in. (the section of a brick longitudinally) is 14 lb. per sq. in. Allowing a factor of safety of 5 to cover irregular bonding we have an allowed tension of 3 lb. per sq. in. Using these values as above for the concrete footing we find

$$h = 3.73\ p.$$

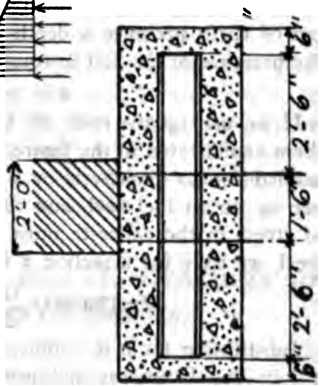
Since the allowed tension on the brick footing is directly proportional to the earth pressure, this relation will hold true for any other earth pressure. It requires good bonding of brick work to distribute the load on a spread foundation, even with the small spread that this formula would give.

From the standpoint of the allowed shear on the con*crete, using* Fig. 10, the amount of pressure on the projec*tion for* an earth pressure of 1 ton per sq. ft. is 2,000 p.



Line representing Grip of rods
Curve of Moments
d
x
h
h/2
p
1/6 h
17/24 h
1/8 h

FIG 12



2'-0"
6"
2'-6"
1'-6"
2'-6"
6"

FIG 11.

The area in shear is 144 h sq. in. Allowing 40 lb. per sq. in. we have 5,760 h = 2,000 p, or

$$h = .35\,p.$$

Sometimes the upward pressure of the soil is resisted by needle beams under a wall, of old rails or I-beams. In Fig. 11 assume the upward pressure of the soil as 2 tons per sq. ft. and the imbedded beams spaced 1½ ft. apart. This upward pressure will be the same as a vertical downward load on the beams, which will be cantilevers having an overhanging of 3 ft. The middle distance of 1 ft. 6 in. is taken less than the width of the wall, as some space is required for bearing. The bending moment on each beam is 1½ × 4,000 × 3 × 1½ = 27,000 ft.-lb. or 324,000 in.-lb. The value of the section modulus required is 324,000 ÷ 16,000 = 20.25. The section modulus of a 10 in. I-beam 25 lb. is 24.4, hence this size of beam would be used. It would take 4 55 lb. rails or 73.3 lb. per ft. to give the same stiffness.

Reinforced concrete slabs are now much used in place of the needle beams shown in Fig. 11. Fig. 12 shows a slab of the kind referred to. We have seen that in order to have a proper shearing area we should have for every ton of earth pressure a depth h equal to .35 p, or if S be the pressure of the soil in tons per sq. ft.,

$$h = .35\,pS \qquad (1)$$

If we use square rods for the reinforcement, and place them as indicated in the figure, we should have the rod imbedded for 50 diameters, or $p = 50d$. Allowing 12,500 lb. per sq. in. on the steel, and remembering that the amount of stress in the concrete must be the same as that on the steel, we have for a section x in. wide and h in. deep.

$$M = 12500d^2 \times \frac{17h}{24} = 8854d^2h$$

Substituting for h its value, .35 pS, we have $M = 3099\,p\,d^2S$ in.-lb. From the soil pressure of S tons per sq. ft. we have

$$M = \frac{2000\,Sx}{144} \times \frac{p^2}{2} = 6.94Sp^2x$$

Equating these two values of M and using for p its value $50d$ we have

$$x = 8.93\ d, \text{ or, say, } 9\ d.$$

It is thus seen that for any upward pressure the same rods would be used in a given projection p. These rods would have a diameter 1-50 of this projection and be spaced 9 times their diameter apart. For different earth pressures the height h would vary as per equation (1).

The foregoing does not take account of the stress on the concrete, but it will be found by trial that the depth of slab required for any earth pressure above about ½ ton per sq. ft. would give concrete enough to keep the stress on the same within safe limits.

In a square slab supporting a column similar reinforcement can be used. The slab should be surmounted by a plinth or block of concrete upon which the column rests. The depth of the slab should be governed by the upward pressure of the soil on all of the area of slab outside of this plinth, allowing a unit in shear (on the section that would be sheared if this plinth should sink into the soil) of 40 lb. per sq. in. Rods should all pass under this plinth; that is, there should be 4 sets, 2 parallel to the sides of the slab and 2 diagonally. Rods parallel to the sides of the slab lying toward the edges and not under the plinth would be of little or no use. They will only serve to intensify the stress on the rods at right angles to themselves that do lie under the upper block or plinth. The use of rods at right angles to each other spaced uniformly both ways is a common but irrational method of reinforcement.

Shear of Concrete and Its Bearings on the Design of Beams.

The unit shearing strength of concrete is between 1 and 2 times its unit tensile strength for practical purposes in the *design of beams.*

In making this assertion the writer appreciates the fac *hat it is a broad* statement. He realizes that it is an a

sertion that will be contradicted. It is not for the mere purpose of raising a controversy that this seeming dogma is put at the head of this article. If the statement is right, it should be given a very prominent place in every work on the subject of reinforced or plain concrete design. If it is wrong, it is the duty of anyone who can show it to be wrong to give a sound reason therefor.

Recently some tests on the shearing strength of concrete were made at the Engineering Experiment Station of the University of Illinois, under the direction of Professor Arthur N. Talbot. These tests have been given a wide publicity. Rightly interpreted they are very valuable. Wrongly interpreted they may lead to another crop of disastrous failures in reinforced concrete construction. The tests referred to appear to show that concrete has a shearing strength nearly equal to, and in some cases in excess of, the compressive strength per sq. in. In other words, they lead to the conclusion that the strength of concrete in simple shear is 8 or 10 times its strength in tension.

In making these tests special efforts were made to eliminate every other stress except simple shear. In the writer's opinion the very means employed introduced elements that to a large extent diminish the value of the tests as helps in design Reference will be made to these features of the tests later. Professor Talbot states that the tests are open to objection. It is the objectionable features that the writer wishes to emphasize.

It is not questioned that the results of the tests give close to the true value of shear in concrete in the straitened conditions in which the concrete of the tests was placed. An attempt will be made, however, to show that the "laboratory" feature of the tests was so intensified by eliminating the beam action, that the results are apt to be very misleading if applied to design. The unit stress allowed in shear on concrete by various building codes is in the neighborhood of 40 or 50 lb. per sq. in. Now if the material has a shearing strength of 1,000 to 2,000 lb. per sq. in., it is a waste of material and an economic blunder

to allow only 50 lb. in design. It would not be surprising if a class of designers should rise up and call for a raising of this low unit to a fair factor of safety based on the high units determined by these tests, especially in view of the high authority from which they emanate. It would further, not be surprising if the work turned out by these same designers should lead to added work for coroners.

One feature of this discussion concerns the difference in the shearing strength of concrete: (1) when in tension at right angles with the plane of shear; (2) when under no stress whatever at right angles with the plane of shear, but free to move in that direction; (3) when under no stress at right angles with the plane of shear, but confined so that any motion would induce such stress; (4) when under compression at right angles with the plane of shear.

It is plain that under condition (1) failure would occur the most readily, and under condition (4) it would be least liable to occur. In fact there would be a large apparent shearing strength between 2 concrete surfaces completely severed, if these surfaces were forced together by a compressive stress, due to the mere friction of the surfaces. This element would be completely lacking in (1), and a small tensile stress, coupled with a shearing action, would cause failure.

Now the conditions in the tests referred to agree closely with (3) and approach (4), while the conditions in actual beams in a building agree nominally with (2) but approach (1). This latter is due to the tendency of a slab to shrink between beams and of a beam to shrink between columns, giving an initial tension throughout the beam or slab. One set of tests was made by punching out a cylindrical piece from a concrete plate or slab. Now if this cylindrical piece had been a separate piece cast in a hole in the plate, with no bond whatever, but merely fitting snugly in the hole, it *would take much* force to push it out. The *apparent but false shearing* strength would be very considerable. *This would come* under the head of condition (3), above

but the actual shearing strength would be zero. Molded in one piece a concrete plate would then have an apparent shearing strength which would be the sum of what actual shearing strength the concrete possesses and this false shearing strength that might be said to be due to the crowding of the material. This would be true independent of shrinking in the concrete. As an illustration of the effect of the crowding of the material, it requires more force to punch a hole in a steel plate when the clearance between the die and punch is small than when it is larger, this in spite of the fact that steel is stronger in tension than in shear.

Shrinking of the concrete block from which the piece is punched adds a new element of apparent shearing strength in that it grips the "punching" and adds, to the 2 other elements that go to make up the force necessary to push it out, that of friction on its sides. That more load was required to force out a punching in a block that was reinforced with straight rods arranged in a square around the tested portion, and still more when hoops were used, is strong evidence of the effect of crowding of the material or of the compression that will be induced normal to the shearing surface by the mere action of the material under the shearing test. It takes a strong pull, sometimes, to draw the cork out of a bottle. There is no element of shear in it, but if this friction were combined with a small shearing strength, the sum would be an apparent shearing strength of no small value.

In another set of tests constrained beams were used. These beams were so short that but little in the way of beam stresses could be expected, especially because the edge of the load and the edge of the support were almost vertically over each other. But the induced stresses of (3) would be very clearly a possibility in these tests. The crowding of the material would add a false element to the *apparent* shearing strength which would be of unknown *value*.

The features of these tests that render the application

of their results to design not only questionable but dangerous are these: (*a*) The test speciments do not resemble anything in which concrete is used practically. (*b*) The loads applied on the specimens, instead of being flexible and taking the shape of the deflecting part, were in themselves rigid for the entire extent of their bearing on the concrete. (*c*) There is scarcely any case in practice where shear, pure and simple, is exerted on any part of a structure in concrete. This last is emphasized in the futile attempts to manufacture a condition where simple shear occurs. In view of the fact that simple shearing action is not a structural possibility, it is not clear why strenuous attempts should be made to approach it in test specimens.

Concrete is unlike steel in that it is 5 or 10 times as strong in compression as it is in tension, whereas structural steel is about equally strong in either. Steel is little affected in its shearing strength by the simultaneous occurrence of tension or compression. The principles of design employed in steel beams and girders must therefore be modified before they can be applied to the design of reinforced concrete beams.

Concrete is further quite different from wood, in that the latter, while it possesses great tensile strength in the direction of the grain, and large compressive strength in the opposite direction, has very little tensile strength at right angles to the grain. The shearing strength of wood in a direction parallel to the grain is small, because of the little tensile strength normal to the grain. If wood were tightly held together with tension bands, its apparent shearing strength parallel to the grain would be largely increased.

About the only common structural material that concrete resembles is cast iron. The tensile strength of the latter in all directions is only a fraction of its compressive strength. The practical value of cast iron in shear is comparatively small because of its weakness in tension, and *yet, no doubt, in* a restrained beam of short span a *block of cast iron* would resist a high shearing strai

The term "practical shearing strength" is here used not only to distinguish from the theoretical but also from ideal.

It is next to impossible to eliminate tensile stresses in concrete beams and slabs, and shear failures will be accompanied in general by tension failures. They will be so closely associated together that it will scarcely be possible to separate them. In fact shear and tension must be considered as generally acting together in beams. A practical working unit stress for shear in beams should take into account the ever present tension. Building ordinances rightly prescribe low units for shear. Reinforcement of beams against shearing stresses, as very often practiced, is not rational.

In Fig. 1 we have a simple beam uniformly loaded. The load in this case is of course flexible and will take the shape of the deflected beam The shear increases from the center of beam to the support. Herein is this beam essentially different from a test beam in which the load is a rigid block the length of the beam and in which the shear is all confined in a narrow space between the edge of the block and the edge of the support. The beam in Fig. 1 represents actual conditions.

The load between *B* and *F* can be assumed as carried on the surfaces *AB* and *FG*. *CD* represents ½ of this load. This may be resolved into *CE* and *ED*. *ED* is the shear along the surface *AB*. *CE* is tension on the same surface. The critical point in the strength of this beam is the tension on the surface *AB*. If this tension becomes too great on this section the beam will fail. It is, of course, a tension failure, but in practice it is interpreted as a failure in shear. In considering the design of a beam it is held to be weak in shear in a section near the ends, because of the known tendency of beams to fail in a line approximating *AB*.

In order to find the value of the angle *X* to give the *tension on A B* a maximum, assume a depth of beam 1-10 *of the* clear opening, as shown in Fig. 1. Assume also a

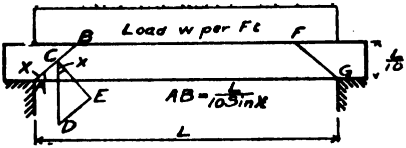


FIG. 1. SIMPLE BEAM UNIFORMLY LOADED.

FIG. 2. COMPOSITE SKETCH SHOWING CRACKS IN BEAMS.

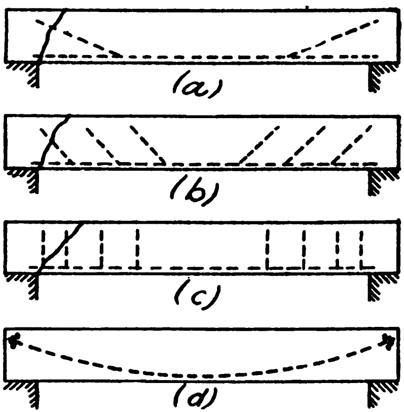


FIG. 3. SHOWING SEVERAL METHODS OF REINFORCING BEAM FOR SHEAR.

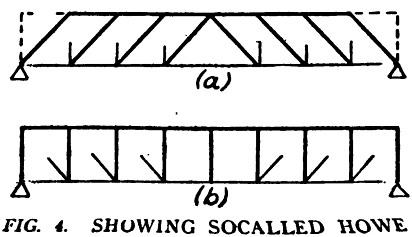


FIG. 4. *SHOWING SOCALLED HOWE & PRATT TRUSSES.*

width of beam of 1 ft. From the trigonometric relations we find

$C E = C D \cos X$.

But $C D = wL\ (\frac{1}{2} - 1\text{-}10 \cot X)$.

Hence $C E = (\frac{1}{2} - 1\text{-}10 \cot X)\ w L \cos X$.

If we divide this value of $C E$ by the length $A B$ (equal area of section in tension), we have

Unit tension on $A B = (5 \sin x \cos x - \cos^2 x)\ w$.

Taking the differential coefficient of this expression for the unit tension and equating the zero we obtain, after reducing

$5 \cos 2 x = - \sin 2 x$.

From which we have

$x = 50$ deg. 40 min.

This is the angle which gives a maximum tension on the section $A B$. From this we have a unit tension of 2.05 w.

The end shear worked out in the ordinary way, is 5 w. Thus in this case the unit strength of the beam in simple shear is for practical purposes 2½ times its tensile strength. It would, in fact, be less than this, because the shear $D E$ in Fig. 1 is acting at the same time, and shear acting in conjunction with tension will lessen the tensile value.

Using Merriman's formula for combined shear and tension, in which the resultant unit tension equals ½ of the direct unit tension plus the square root of the sum of the square of the unit shear and ¼ of the square of the direct unit tension, we find for this case a resultant unit tension of 3.73 w. Comparing this with the unit shear found in the ordinary way (the reaction of beam ÷ area of vertical section, or 5 w), we find that the unit strength is for practical purposes 1 1-3 times the tensile strength.

If, instead of making the depth 1-10 of the span, it be made 1-20 of the span, we find after a process similar to the above, $10 \cos 2 x = - \sin 2 x$, or $x = 47$ deg. 51 min. *The unit* tension is found to be 4.53 w. The unit shear, *found in* the ordinary way, is 10 w. Hence the unit shear-

ing strength is, for practical purposes, about 2¼ times the unit tensile strength, or about 1¼ times, when reduced for combined shear and tension. Most beams will be found to be between 1-10 and 1-20 of the span in depth. It would be found that in beams less than 1-20 of the span in depth this apparent shearing strength approaches closer to the tensile strength, but it is in deep beams where the shearing strength plays the most important part. As pointed out in an article in this journal, published Jan. 15, 1907, the critical depth of beam from the standpoint of shear is 1-10 of the span and over. The capacity of a shallow beam in bending is not sufficient for it to carry load enough to tax its capacity in shear.

Other forces are of course at work in the beam, and these may tend to alter the line of failure, so that it would not coincide with *A B* in the figure. Tension in the lower part of the beam begins to make itself felt a little away from the support. In beams where the bottom reinforcing rods stop at the support, and therefore lack anchorage, the combination of this flange tension and the diagonal tension may reach a maximum a short distance from the support. If a beam is continuous or restrained at the support, there will be compression at the bottom of beam near the support, and this too will tend to make the line of failure or weakness some distance away from the support.

Fig. 2 is a composite sketch showing the cracks in a set of 10 beams tested by the writer. The diagonal cracks are very plainly characteristic and show clearly the weakness of reinforced concrete beams in this respect. These beams were reinforced with straight horizontal rods near the bottom and with rods across the support to take the continuous action. They were also reinforced with so-called shear bars, that is, small vertical bars or rods strung along the reinforcing rods

In "Proceedings of the American Society for Testing Materials," Vol. IV, p. 498, Prof. F. E. Turneaure describes some test beams which were 6x6 in. and 60 in. in *span, reinforced with* stirrups, spaced 3 in. apart. On

page 507 Prof. Turneaure says: "In but a few cases was the failure free from the influence of shearing stresses, but the rupture usually occurring outside of the load and on a diagonal line." The stirrups referred to are supposed to reinforce the beam for shear. As will be noted, these test beams were of a depth 1-10 of the span. They were therefore of a depth where shearing stresses begin to play an important part.

Another characteristic in the behavior of the beams represented in Fig. 2 is the horizontal crack just above the reinforcing rod These beams were too narrow and were consequently lacking in shearing strength, in a plane just above the rods, to take the horizontal shear or the gripping force of the rods. This corresponds to horizontal shear in the web of a girder, the stress that the flange rivets must convey from web to flange. The beams of which this sketch is a composite were designed as T-beams, including in their calculation the floor slab. Thus the lower part of the beam did not have enough concrete to take care of the stresses in the steel. When it is considered that the load upon the beams was only that which the designer intended should be the safe carrying capacity, the T-beam is seen to be weak on account of a narrow stem that does not contain enough concrete to take the horizontal shear above the rods. Some of these beams, under the test load, showed only vertical cracks starting at the bottom of the beam and taking in the reinforcing rod. One beam had 16 of these cracks in its length, another evidence of the weakness of a T-beam.

The shearing stress just above a horizontal rod in a reinforced concrete beam does not have the assistance of any initial compression to relieve the effect on the concrete. There is, further, not any crowding of the material to make failure less liable. Reinforcement with short diagonal or vertical rods either attached to the horizontal rods or looped over them, could only take this shear by localizing or concentrating the stress at these rods and introducing a new element of weakness.

It is a remarkable fact that if we apply Merriman's formula for combined shear and tension and make the tension zero, we find that the tensile unit stress due to shear alone is equal to the shearing unit stress. It would follow, then, that in the case of horizontal shear above reinforcing rods the shearing strength is equal to the tensile strength of concrete. In the light of this fact a T-beam having horizontal rods, with its narrow stem and large percentage of steel, is a most absurd and inefficient, as well as unscientific, form of construction.

Fig. 3 shows several ways of reinforceing a beam for shear. That shown at (*a*) is faulty on account of the sharp bend in the reinforcing rod (the one bent up toward the support). Failure could occur on the diagonal line by simply pulling a short length of rod out of the concrete. At (*b*) and (*c*) failure could occur by the pulling out of a short end of one of the "shear rods." Explanation of how these shear rods act to reinforce a beam in shear seems to be lacking in technical literature. A rod, to be effective in taking stress, must either be anchored at the end by a nut and washer, or some equally effective means, or must be buried in concrete for some distance beyond its point of usefulness. It is conceded that a mesh of overlapping rods will be some aid in resisting shearing stresses. As an economic proposition, however, this means of reinforcement is a failure.

A steel rod in shear, that is, shear in the steel, stress at right angles to the axis of the rod, is an absurdity. Concrete would not stand the side force of a rod against it that is stressed in shear to anything like a reasonable safe value in the steel.

In Bulletin No. 12 of the Illinois Engineering Experiment Station the results of some tests on T-beams are given, which illustrate shear reinforcement with vertical rods. In the beams the tests of which are there described all of the portion of the beam in shear, namely, the end third at each end, is reinforced with ½ in. U bars 6 in. apart. There is about 200 in. of ½ in. corrugated bars in the stirrups alone

in a beam of 10 ft. span. This steel would weigh 14 lb. If the 4 ¾-in. plain round rods found in one beam were curved up and run through a washer plate at ends of span, 2 nuts on each end of each rod and 2 washer plates 5-16 in. thick and 7 in. by 5 in. would not weigh as much as these stirrups. Further, these parts would not be of special steel. Such a detail would give a positive and scientific provision against shear. In a line of continuous beams rods curved up add scarcely any length or cost. Commenting on these tests it is significant to note that the unit shears given by Professor Talbot as existing at first diagonal crack run from 180 lb. to 302 lb. per sq. in. In the writer's opinion these show where the concrete failed in shear, or tension due to shearing action. After this failure of the concrete the stirrups were brought into play, as it would be generally necessary for some "give" to take place in the concrete before the U bars would have a working bearing on the horizontal reinforcing bars. This is equivalent to reinforcing a detail in steel work with a bent plate that will come into action after the main detail fails or slips. Of course a beam is stronger with a system of shear bars in close spacing than it would be without. The writer believes, however, that shear bars are neither an economic nor a scientific method of reinforcing a beam.

None of the tests in Bulletin No. 12 failed by shear, and some of them stood double the load required to give the first diagonal crack due to shear. It is a precarious sort of construction, however, that entails a system of cracks in the concrete before it can be brought into play. A load that would produce the first crack in the concrete would not be a safe load on a beam. If a given load causes cracks in a beam it can scarcely be said to be in any better case under a great number of repetitions of that load, than is a member subjected repeatedly to stresses nearly equal to the elastic limit. We know that the latter would eventually cause failure.

It is common to hear the system of horizontal bottom flange rods and vertical or diagonal shear rods spoken of

as a truss system and to hear comparisons made with a steel Pratt truss or a wooden and steel Howe truss. Now, since, as stated, the shear rod must have anchorage of some sort outside of the point where it can take any stress, we can represent these so-called Howe or Pratt trusses as shown in Fig. 4 at (*a*) and (*b*) respectively. The heavy lines represent the compression members, or in other words, the concrete acting in compression. The light lines represent the tension members, or the shear rods. These are good from the point where they are attached to the rod to the point beyond which they have an efficient anchorage. It does not take much imagination to see what would happen to a truss on the lines of either of these. And yet these are true representations of just what we find in reinforced concrete design as used in practice and as sanctioned in some of the best works we have on the subject. These abbreviated tension members must of necessity be aided by tension in the concrete, the most unreliable factor in its strength.

Fig. 3 at (*d*), shows a beam reinforced for shear in a manner that is rational and meets all requirements. If this beam were cracked, the rod could not pull out because of the anchorage. If the beam be one in a continuous line, the rod could pass into the next span for anchorage, acting in that span at the same time as reinforcement for the top flange to take the stress due to continuity. This sort of reinforcement is needed in beams less than 1-10 of the span in depth.

To return to a discussion of the tests made at the University of Illinois. There are some features about the results of these tests that it is instructive to note. Average values do not count for much where there is a wide range of results. For example, if there is a difference of 100 per cent between the lowest and the highest result, and if the mean is midway between these, a design made on the basis of the mean value with a factor of safety of 4 will have *a factor of safety of* less than 3 on the basis of the lowest *value. Often there* is a difference of several hundred per

cent in the result of tests on such uncertainties as bending and shear in wood, shear in concrete, etc. Designs on the basis of mean values may thus have a factor of safety that is half fictitious, for the lowest value is as apt to be found in the finished structure as the highest or as the mean.

Among the punching tests on plain plates there is one that shows a shearing strength, at first crack, of 142 lb. on 1:3:6 concrete. One test on 1:2:4 concrete shows 213 lb. per sq. in. at first crack. One recessed block of 1:3:6 concrete shows 258 lb. per sq. in. at first crack, and one of 1:2:4 concrete shows 332 lb. One reinforced recessed block of 1:3:6 concrete shows 496 lb. per sq. in. at first crack and one of 1:2:4 concrete shows 668 lb. One restrained beam of 1:2:4 concrete shows 783 lb. per sq. in. at first crack. These results run from ½ to ¼ of the ultimate shearing strength found. It may be true that all of these low values were the result of tensile stresses due to beam action. If this is true in a test where every effort is made to eliminate beam action, how can anything else be expected in practice?

Turning now to the ratio between ultimate shear and ultimate compression we find it to be as low as .37 for plain punched plates, .39 for recessed blocks, .67 for reinforced recess blocks, and .44 for restrained beams. In some tests the apparent ultimate shearing strength was more than the ultimate compressive strength. This wide variation shows the uncertainty in shearing strength of concrete, to which reference has heretofore been made.

A fair average value for tension in 1:2:4 concrete is about 200 lb. per sq. in. Using 1 1-3 times this as a basis for shear 40 lb. and 50 lb. per sq. in. are seen to represent factors of safety about 6 and 5 respectively. These factors are not any too large, in view of the uncertainty exhibited in concrete in shear and tension, for use in the design of beams.

The Design of Reinforced Concrete Columns.

The strength of a plain concrete column is largely determined by the shearing strength of the concrete. In Fig. 1, at (*a*), is shown a column carrying a load *P*. If this be resolved into *C B* and *C A*, the latter will be compression on the surface *M N* and the former will be shear on the same surface. The angle *y* to make the unit shear on *M N* a maximum will be 45 deg. and the intensity of that unit shear is just half the unit compression on the column. Two planes of failure may meet as at (*b*), forming a wedge. The point of the wedge may be crushed, or the wedge may act to split the column. This bulging or splitting action is a common failure in wooden columns, on account of the weakness of wood in shear. It is also a typical form of failure in concrete columns. The wedges or cones may form at the ends and split or bulge the entire column. At (*c*) is shown the futility of reinforcement with longitudinal rods of small diameter. These will bend if the column bulges, and will be of little use. In cubes under test the cones will meet, either apex to apex or base to base, and

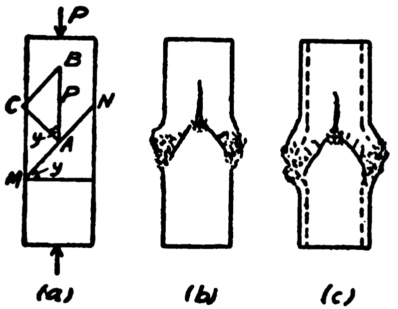


FIG. 1. SHOWING BULGING ACTION IN COLUMN.

only *corners will be* flaked or spawled off by the shear, *leaving the core in* compression; hence the fallacy of re-

lying on tests on cubes to determine the compressive strength of plain concrete shafts or columns.

In laboratory tests, plain concrete columns of 5 or 10 diameters in length will sustain moderately high compressive stresses. This is because of the centric application of the load. The shearing strength of concrete in a plane which is at the same time subject to compression is very much greater than if there is tension instead of compression, acting on that plane; it is also greater than if there is no stress normal to the surface. The friction between the surfaces due to the compressive strength adds an apparent shearing strength to the actual shearing strength of the concrete. It is also true that columns reinforced with longitudinal rods, in laboratory tests, sometimes show moderately high compressive strength. This may also be attributed to the fact that the load is applied exactly in the center of the column.

Only a small shifting of the load on a plain concrete column, or one reinforced with longitudinal rods, is necessary to bring into play the weakness of the concrete. When the center of gravity of the applied load on a square column is more than 1-6 of the diameter from the center, or when that on a circular column is more than ⅛ of the diameter from the center, there will be tension on the extreme fiber of the column. The shearing strength, which is the critical factor in the strength of the column, is immediately weakened when the compression is removed and both shear and compression become of double intensity on the other side of the column. The bulging action illustrated in Fig. 1 will be augmented by eccentric loading.

Eccentric loading of columns in buildings is the rule and not the exception. In tests it is the rule to apply loads centrally. It is of utmost importance that reinforced concrete columns be made capable of resisting eccentric loading or flexure.

Concrete columns with longitudinal rods embedded in *them* are not efficient and are not rational design, for the *following* reasons:

(1) The column is a composite of steel and concrete, and not a true reinforced concrete column, as the steel must be chiefly in compression.

(2) There is no way to determine, even approximately, the respective amounts of the load that the steel and the concrete will bear. The use of the moduli of elasticity of the two materials is no aid, on account of the fact that the concrete tends to shrink and shorten, leaving the steel rods longer than the normal unstressed concrete.

(3) If the rods are near the surface of the concrete, they can more easily break out under the bowing action of direct compression or the bulging action of diagonal shear.

(4) If the rods are near the center of column, they will be of no aid to resist flexural stresses due to horizontal forces or to eccentric loading.

(5) Longitudinal rods offer little or no resistance to longitudinal splitting or bulging of the column.

(6) The rods cannot take diagonal shear without overstressing the concrete.

The assumption that they can take shear, in amounts of anywhere near the capacity of steel to carry shear, is simply untenable and absurd, in spite of recognition in building codes and regulations. If, for example, we assume a shear of 12,000 lb. in a rod 1 in. sq., there must, of necessity, be a bending moment in the rod. Now the square rod, at 24,000 lb. extreme fiber stress, would take a bending moment of 4,000 in.-lb. The lever arm of the 12,000 lb. would then have to be only 1-3 of an in. The force of 12,000 lb. applied on the side of a square rod in a length of 1-3 of an in., or on several times this length, is beyond the power of concrete to withstand.

(7) A plane of cleavage, especially if it be a sloping one, such as a joint left where pouring of concrete ceases for a while, will leave a weak section and vitiate to a large extent the factor of safety.

Comparison of a reinforced concrete column with a steel column as a basis of design is misleading, because of the fact that steel is very strong in tension and therefore capable

of resisting bending stresses. Cast iron columns were formerly proportioned on the basis of 11,300 lb. per sq. in. (reduced for length). Full size tests made some years ago (see *Eng. Record,* June 11, 1898) showed this unit to be too high and that a proper unit is about 7,600 lb., reduced for length of column. The compressive strength of cast iron in short blocks is about 100,000 lb. per sq. in., but on account of the low tensile strength, and consequent low shearing strength, the safe unit in columns has but a remote relation to the compressive strength in short test pieces. An exactly similar condition exists in columns of plain concrete or of concrete that is not reinforced, with a view of relieving it of all tensile strains and of excessive shearing strains.

Practical experience has proven the inability of concrete columns in which small rods are embedded to carry heavy loads. The practical experience referred to is the failures of buildings that have recently occurred.

When a small section of a building falls, the failure may be attributed to a local cause, generally a fault or defect in the beams. When a general collapse of a large section of a building occurs, it is probable that the failure is due to weakness in the columns. If one beam fails, it ought not to bring down with it much of the rest of the structure. When a column fails, it lets down a large section of the floor; and if the other columns have the same weakness they will follow. The great weight falling on lower floors will overload these by the shock and add to the extent of the failure.

In a failure that occurred some years ago, the reinforced concrete roof of a large building fell in on account of the concrete not having set before the forms were removed. The weather was very cold, and the concrete froze instead of setting. This great weight falling did not affect the floors below and did not affect the columns supporting them. The columns had steel spirals and longitudinal rods wired to them. The beams and girders, while not ideal in design, had rods across the supports to tie them together.

Recently two large buildings being constructed exhibited such serious weakness that large sections of them collapsed. The columns of these buildings were made of concrete with some longitudinal rods through them. The safe strength of such columns cannot be said to be intimately related to the compressive strength of concrete in cubes; and yet the unit compression, in some cases, for dead load alone, was in the neighborhood of ¼ of the ultimate compressive strength of concrete cubes. Compare the ultimate strength of cast iron with the unit used in a properly designed column.

Square concrete columns in which longitudinal rods are placed near the corners, and these rods tied together by straight rods or bands, are a step in the right direction. These horizontal ties will resist the bursting or bulging tendency of the load; but as this tendency is in all directions, it will act to make the square formed by these ties into a circle. The outward force at the side of the square is not adequately resisted. The metal is not economically disposed. It is somewhat analogous to a square vessel containing a liquid. A round or cylindrical vessel will require much less metal and will be more rigid.

It follows, then, that rational design of reinforced concrete columns demands not only longitudinal reinforcement to take flexural stresses, but circular reinforcement to take the bursting or bulging forces due to diagonal shear. Columns so designed have proven under test to be the strongest of all known forms of reinforced concrete columns.

A steel cylinder filled with concrete would meet the requirements most completely. Concrete will stand very great pressure thus confined, even to the extent of being forced out of shape and still retaining its adhesion. Columns so designed, however, do not seem to be suitable for ordinary construction.

Flat bands in hoops or in a coil, having a diameter *somewhat less than the* column, so that they will be *surrounded by concrete, in* conjunction with longitudinal rods.

would seem to be a near approach to the cylinder filled with concrete. Concrete, however, will not grip flat bands as it will round and square rods and will not adhere as well to them. There will be the tendency of the concrete to break off outside of the bands, especially if the space between bands is small. Columns thus reinforced will take very heavy loads.

A practical objection to bands for reinforcing hoops or spirals is that it is more difficult to puddle the concrete around them and make it fill the voids outside of the bands. Another objection is the greater liability of concrete to break off the flat bar under the heat of a fire and thus expose the steel to the heat. Bands would have to be welded, and a weld in steel is more or less of an uncertainty.

A good and efficient column is made by reinforcing a round or an octagonal column with a coil made of a square rod and with 8 longitudinal square rods wired to the same, just inside of the coil. The purpose of the longitudinal rods is to take flexural stresses, that is, to relieve the concrete of longitudinal tensile stresses due to any side force or any tendency to bow at the middle of the height of the column. The steel thus used is rationally employed, as it takes tension that would otherwise come on the concrete. These steel rods should not be counted upon to take any of the direct load of the column, because of the fact that tests show that such rods alone in a concrete column offer little or no assistance to the concrete.

When a concrete column is under compression, its length is diminished and its diameter is increased somewhat. The steel coils come into play by this tendency of the column to increase in diameter and are therefore in tension.

In a series of tests made by M. Considère, hooped concrete prisms showed very high compressive strengths. They further showed a regularity in the matter of breaking. Plain concrete prisms broke suddenly, whereas hooped prisms would hold together after partial failure had occured and would sustain great loads in this condition. The immense advantage of this toughness in a column is very

nanifest, and it is the strongest argument for hooped colımns. It is plain that partial failure in a column with only ;mall longitudinal rods in it would be a matter of very ;erious moment. The possibility of shrinkage cracks in :oncrete must always be kept in mind. Longitudinal rods n a column offer no safeguard whatever against such :racks, whereas in columns reinforced with a spiral or ıoops in addition to longitudinal rods, cracks are of no ;erious consequence.

Tests made at the Watertown Arsenal in 1906 on colımns 10 to 12 in. in diameter and 8 ft. in height, corrobo·ate the results of the tests made by Considère. They show, n general, that as hoops approach each other the ultimate ,trength of the column is very greatly increased.

A basis for determining the size of the rods in the coil s found in analogy of the column to a tube containing a iquid under pressure. The stress in the rod corresponds o the annular tension in the walls of the tube. Under the ıltimate pressure the concrete would be in a disintegrated tate and would resemble sand. In this condition it would :xert a lateral pressure, but not a pressure equal to that :xerted by a liquid under the same force. M. Considère 'ound that the lateral pressure was 10-48 of that of the ongitudinal pressure in disintegrated concrete and in sand.

If, at failure, there is a lateral unit pressure correspondng to 10-48 of the compression in the direction of the axis ›f the column, we can assume that the same relation holds rue under safe loads. Whether or not it does hold true is ›f little consequence, so long as it is certain that this asumed stress in the steel is not exceeded, it is a sound basis or finding the size of coil for the following reason:

Suppose we have a column which, under tests, has disntegrated so as to be practically like sand in the matter of :xerting lateral pressure. The lateral unit pressure will ›e 10-48 of the unit compression, and this, on rods strained o their ultimate usefulness (the elastic limit, as usually *onceded in reinforced* concrete work), would give a con-*tion where the* concrete and steel are consistently propor-

tioned. Now, if a factor of safety be applied, both to the stress in rods and the compression in concrete, the consistency of the design is maintained, though the actual stress in the steel is not thereby definitely determined. Suppose that it were known that the rods in some construction were not stressed much until 9-10 of the ultimate load were upon it, and that they then suddenly received their load. We would not have a consistent design unless the ultimate strength of the rods corresponds with the ultimate strength of the structure, in spite of the fact that the actual stress in the rods under a safe load on the structure may be small and indeterminate. It is conceivable that in a reinforced concrete beam the concrete, in some cases, takes nearly all of the tension up to its ultimate strength, and that in the case of stress above its ultimate strength the stress at the crack is imparted to the steel in its entirety. This may be the explanation of the fact that beams sometimes show no cracks on the tension side under stresses which, if concentrated in the reinforcing steel, represent elongations that would overtax the integrity of the concrete. Granting for the sake of argument that this is the case and that under safe loads, with perfect concrete, the steel is under very little stress, and that the concrete in tension is doing practically all of the work; it can by no means be construed as warrant for proportioning the steel for this ideal condition. Laboratory tests, both on beams and columns, lead to conclusions such as this, because of the ideal conditions that prevail in the manufacture and testing of specimens.

Practical design must take into account actual conditions in practical construction and practical defects that are liable to be incorporated in a structure.

In a hooped column or one reinforced with a spiral, a condition may exist that is analogous to that just postulated of a beam. It is therefore in reason to proportion *the steel* in a column as though it took all of the tensile *stress tending* to bulge or burst the column, just as we *proportion* the steel in a reinforced concrete beam to take

all of the tensile stresses that might come on the concrete.

An assumption, such as that made in a recent work on reinforced concrete, namely, that a column has a compressive strength due to the strength of the concrete alone in compression, and another compressive strength due to the hoops or spirals, is untenable. Both of these supposed joint strengths would break down at once. They are simply different phases of the same thing. The plain concrete alone would be weak as a column, and the hoops or spiral alone would be absolutely useless as a column. The hoops supply what the concrete needs to hold it together. The concrete between the hoops is simply plain concrete. No quality is imparted, by the reinforcement, to the concrete between the coils that it does not possess in short blocks or discs. Hence a proper unit load would be a safe value for compression on short prisms of plain concrete. If the concrete be the standard 1:2:4 concrete generally used in reinforced concrete work, a proper unit in compression would be between 500 and 600 lb. per sq. in.

It is true that in columns where the hoops are spaced close together the compressive strength of columns under test runs up to 5,000 and 6,000 lb. per sq. in., just as thin discs of concrete would show correspondingly high unit strength. Close spacing of hoops, however, entails practical difficulties in puddling the concrete around the bars, and the almost continuous cylinder of metal forms a cleavage surface from which the concrete is apt to break away. This last might result from ordinary changes of temperature, and would be a strong probability in the case of a fire.

In the work on reinforced concrete above referred to, the author, while finding a compressive strength in a set of loose hoops, or a flimsy coil, apart from that of the concrete which they are intended to reinforce, and in addition to the strength of that concrete, shows his lack of confidence in his formula by throttling his unit value with an *empirical constant that* almost cuts it in half.

If we assume a safe load of 550 lb. per sq. in. and a

lateral pressure of 10-48 of this in intensity, we have a basis for the determination of the tension on a coil. Let the pitch of the coil be 1/8 of the diameter of the column and let

D = diameter of column in inches;

d = diameter of square steel rod in the coil, in inches.

Equating the equivalent fluid pressure on the rod to its tension at 12,500 lb. per sq. in., we have

$$550 \times \frac{10}{48} \times \frac{D}{2} \times \frac{D}{8} = 12{,}500 d^2$$

Solving we find

$$d = \frac{D}{42}$$

If we make the diameter of the coil 7/8 of that of the column, and the diameter of the square rod of which the coil is made 1-40 of the diameter of the column, we shall have close to 12,500 lb. per sq. in. on the steel.

For the 8 rods which run the length of the column we may assume the same lateral pressure and proportion the rods to take that pressure. Assuming that they would act to resist the outward pressure of the disintegrated concrete, at the ultimate strength of the column, we can make the rods of a diameter that they would take the stresses in bending, at a safe unit, due to a lateral pressure 10-48 of 550 or 115 lb. per sq. in. The outward force per inch in the length of rod is $115 \times \pi \times D \div 8$. The clear span is 1/8 of D less 1-40 D = 1-10 of D. As the rods are fixed ended, the bending moment is 1-12 of $w\,l^2$, or

$$M = \frac{115 \pi D}{8} \times \frac{D^2}{100} \times \frac{1}{12}$$

Equating this to

$$12500 d'^3 \div 6$$

the resisting moment of a square rod of a diameter d' we obtain

$$d' = \frac{D}{38}$$

As this is close to 1-40 of the diameter of the column, we

may use the same size of square rods as that used in the coil.

It is recommended, therefore, that reinforced concrete columns be made round or octagonal and that the entire area of the circle or octagon be considered as taking the load; also that the reinforcement be made of a coil of square steel rods of a diameter one-fortieth that of the column; also that just inside of this coil eight rods of the same diameter be wired to the coil. At the end of a coil the rod should lap a half a circle, as this would be about 55 diameters.

The unit of 550 lb. sq. in. would be used for lengths up to 10 diameters. Between 10 and 25 diameters the allowed unit pressure would be found by the following formula:

$$p=670-12\frac{l}{D}$$

where $p=$ allowed pressure per sq. in.,
$l=$ length in inches.
$D=$ diameter in inches.

Columns more slender than 1-25 of their length should be avoided. The same reinforcement should be used in all columns of a given diameter, so that flexural and eccentric stresses will be taken care of in long columns.

The practice of using such units as 1,000 lb. per sq. in. on concrete in columns is fraught with great danger. Laboratory tests on carefully prepared specimens under perfectly central loading are not proper criteria for the design of columns that have to take eccentric loads and in addition have to resist the wind loads tending to sway a building. A quality in columns that is very essential, especially in high buildings, is toughness. High buildings of 12 to 15 or more stories require very large columns in the lower stories, at units that are proper for concrete; hence some other kind of column may better be employed for the stories where very heavy loads are carried. A suitable column would be of steel latticed on two sides and filled and surrounded with concrete. Such a column

Another kind of steel column coul
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concrete beams and girders can pa
gles could be connected by batten
a column, while it is objectionable
on account of having no web to t
part of the column to another, wo
if it were completely surrounded w

Steel columns such as those refe
into reinforced concrete columns
the latter to pass down over the
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For a unit load on a steel colum
with concrete a value much highe
column of the same section coul
Units between 14,000 and 16,000 lb
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Design of Dams and Use of Concrete Therein.

Structures for the impounding of water are among the oldest of engineering works. The forces acting upon dams are in many ways among the simplest and most easily determined. It is a sad commentary on modern engineering to say that some of the worst calamities in which man was a causal agency have been the result of the overthrow of structures whose stability is a matter of the most elementary calculation and the forces against which are completely determinate.

The writer makes this preface to the present article in order to emphasize a factor that he believes to be the cause of many failures of dams, and because in all technical literature there is scarcely a mention made of this factor. The factor referred to is none other than the floating or buoyant power of the impounded water. An offhand answer to this would be that water will not tend to float anything not submerged. It is practically impossible to seal the up-stream side of a dam against the admission of water under the dam, and the thinnest sheet of water, entering the finest crevice, exerts the same uplifting tendency on the under side of a dam that would be exerted in a large crack. Even if this upward pressure on the under side of the dam be diminished uniformly to zero at the down-stream edge of the dam, where the water emerges, the effect of the pressure on the stability of the dam is the same as though the pressure of the full head were maintained for the entire width of the dam. It is true that this pressure can be diminished by under-drainage, but this would be apt to waste much water, and it is seldom resorted to.

Much has been said of the suction exerted on the down stream face of a dam when water is spilling over it, in attempted explanation of failures. It is a dangerous theory that magnifies *non-essentials* and overlooks essentials. *There is a small negative* pressure under the falling sheet *of water, probably amounting* to as much as a strong wind,

in some cases, or even, in large dams, to a foot or two of water on a portion of the face. But it has never been shown that this negative pressure actually equals the drop of the water head from the level of the main body to a point vertically over the crest of dam. Now it is this head of the main body of the water that should be used in proportioning the dam; and, as the hydrostatic pressure on the back of the dam is measured by the head at the crest, there is this margin of the difference between these two heads that will be ample to cover both the negative pressure on the face and the effect of the small flow in the main body. The effect of the flow is to add pressure on the back of the dam above that which would be produced by the static head just over the crest, but, as only the upper strata of the water are in motion, the water has a velocity head or dynamic head only to a very limited degree.

Impact of water on a dam is another element whose importance is probably overestimated. For a large body of water to impinge on the back of a dam it would have to advance with a front nearly vertical into an empty reservoir. This would not even be approached except by the failure of a dam at a higher elevation. Floods have the natural effect of increasing the head and the static pressure on the back of the dam, and if the maximum level of water is known and the dam designed therefor, there should be no more fear of a dam failing than of a railroad bridge failing when it receives its full calculated maximum load.

The foregoing is not written to show that the design of dams is the simplest of operations. There are many difficult problems in connection with their design, especially in such construction as earth dams and rubble dams. The greatest problem, however, is generally to come within the appropriation, and too often this is done by ignoring the most potent factor opposing the stability of the dam.

In the design of dams the factor of safety, with which we are familiar in other branches of designing, has scarcely any meaning, at least as regards stability against overturning. With those structures where the factor of safety

plays an important part there can not be said to be a point where a structure ceases to be safe and becomes unsafe or vice versa. In stability against overturning, however, there is a distinct line on one side of which a structure is stable and on the other side of which it is unstable. There is a popular notion, owing its existence to public school "philosophy," that an object cannot be overturned until the center of gravity falls without the base. This is quite true of miniature objects, but a great wall of stone stood upon its corner would be disintegrated by the heavy pressure on that corner. With all possible external pressures considered and the resultant of these pressures and the weight of the dam falling within or at the edge of the middle third of the base, the dam will be stable. If the resultant pressure falls without the middle third of the base, the dam will not be stable, as it will have a tensile or uplifting force sufficient to overcome its own weight at the upstream edge whenever the extreme condition is reached. This rocking of the structure is a distinct condition of instability. Opening of the joints on the up stream side of the dam is a menace to its stability, because it allows water at high pressure to get beneath the masonry. This water acts as a wedge to penetrate further. If the dam is not calculated to resist this lifting or buoyant pressure, its stability is seen to depend upon the sealing of the upper face against the admission of the water. Leaking of reservoirs is too common an occurrence to need any emphasis. That dams should be built, as they sometimes are, whose stability depends upon their watertightness, seems incredible, unless it is the result of ignorance. This latter assumption is probably not far from the truth in view of the paucity of information in technical works on the subject heretofore alluded to.

Barring dams that are built in the form of arches, that is, with a horizontal curve in plan, dams are usually built of a few general shapes. As the weight of water is uniformly about 62.5 lb. per cubic foot and that of masonry is *generally about 150 lb.* per cubic foot, it is a simple matter to determine a ratio of base to a height that will

resist all possible water pressure. It is also a simple matter to try any dam of known dimensions and see whether this ratio obtains.

In order to judge of the stability of a dam of ordinary cross section the ratio of depth to height will be determined for a dam of rectangular cross section capable of resisting only the horizontal pressure against the wetted face, neglecting in this calculation the pressure on the under side of the rectangle.

The unit pressure of water in every direction is the same at a given depth, and in amount is equal to the weight of a column or prism of water whose cross section is unity and whose length is the depth below the free surface of the water. Another principle of the pressure of fluids is that it is normal to the surface against which it is exerted. The determination of the stability of a dam amounts, therefore, to finding its ability to resist easily determined forces. Assuming w to be the weight per cubic foot of water, the horizontal pressure per square foot on a vertical wall at a depth h below the surface of water is wh. The pressure increase directly as the depth, so that the triangle of Fig. 1 represents the forces acting upon the wall or dam. The center of gravity of this pressure is one-third of the height from the base, and the amount, for one foot of length of the dam, is the area of the shaded triangle, or $wh^2 \div 2$. The overturning moment about the base of dam is

$$\frac{wh^2}{2} \times \frac{h}{3} = \frac{wh^3}{6}$$

This moment must be resisted by the weight of the dam, and in order that there be no tension on the masonry the resultant of the horizontal force and the weight of the dam must fall within the middle third of the base. The lever arm of the weight of the wall must therefore not exceed one-sixth of the base. The moment of stability for one foot of length of the dam is then $Wb^2h \div 6$, where W is the weight per cubic foot of the masonry. Equating the overturning moment and the moment of stability, and solving we have

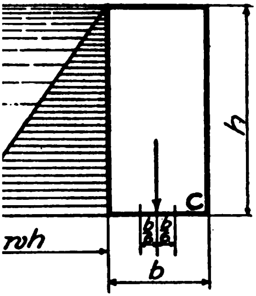

FIG. 1.

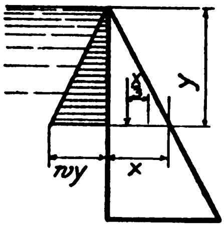

FIG. 2.

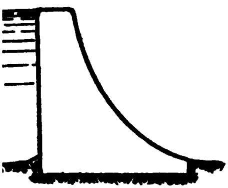

FIG. 3.

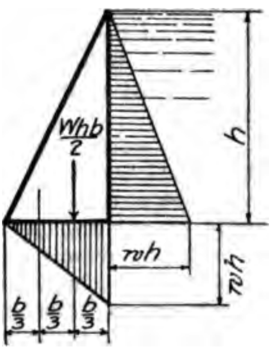

FIG. 4.

$$\frac{h^2}{b^2}=\frac{W}{w}$$

Using 150 and 62.5 as the weights per cubic foot of masonry and water respectively we find that the base should be .65 times the height.

A dam just capable at any horizontal section of resisting the static pressure of water whose surface is at the level of the top of the dam would be triangular in section. In Fig. 2 we have for the equation between the overturning moment and the moment of stability the following:

$$\frac{wy^3}{6}=\frac{Wx^2y}{6}$$

From which we have

$$\frac{y^2}{x^2}=\frac{W}{w}$$

Hence the width of the dam at the base should be about .65 times the height. This is seen to be the same as for the rectangular wall, though the amount of masonry is only one-half as great. The reason that the moment of stability of the triangular wall is as great as that of the rectangular wall is because the center of gravity of the triangle that is cut away is directly over the center of moments.

A dam would of course not come to a point at the top but would have some width. Masonry added to increase the width at the top is well placed, as it increases the moment of stability. A form of cross section very often used has the down stream face curved, as in Fig. 3. This does not waste masonry in the third of the base where weight does not count for anything in stability. This toe of the dam must, however, be strong enough to resist the forces induced by the tendency to overturn.

Turning now to a consideration of the proportions of a dam that will resist not only the horizontal pressure of the water but also the upward pressure from beneath, we have a representation in Fig. 4 of the forces acting on such a dam. We might assume that the pressure of the water neath the dam gradually diminishes as it works its way

toward the toe of the dam. The lower shaded triangle would represent the upward forces against the base of the dam. It will be seen that the moment of a rectangle of altitude *wh* about the center of moments is the same as that of the triangle shown, hence it is immaterial whether or not we assume a diminution of pressure.

The total overturning moment equated to the moment of stability gives us.

$$w\,(h^2+b^2)=W\,b^2.$$

Using the same weights as before for water and masonry we have

$$\frac{b^2}{h^2}=\frac{5}{7}.$$

The width of the dam at its base should therefore be about .85 times the height. Of course making the cross section of the shape shown in Fig. 3 will increase the moment of stability, and the base need not be quite so wide. On the other hand if a height of water greater than the crest of the dam is anticipated, as is very often the case, the above relation would not be sufficient.

This ratio of height to base is derived merely to show what should be the approximate relation for a solid masonry dam capable of resisting all of the possible forces against it. There are not many dams that have a width of base .85 of their altitude and there are not many that are so thoroughly underdrained as to make this width unnecessary. A builder may take long chances on the probability of an office building or a highway bridge never receiving its load and be comparatively safe. But the pressure of water is something sure and determinate, and the integrity of a dam is something that ought not to be trifled with.

If the slope of a solid dam be made on the upstream face, while the pressure of the water will act partially to prevent overturning, the moment of stability of the masonry is neutralized. As the water is of less weight than the masonry, the resultant stability is greatly diminished. If the slope of the upstream face were made 45 deg., the *resultant pressure* of the water would fall at the edge of *the middle third.* Such a dam, thoroughly underdrained

would not need to depend upon the weight of the masonry for its stability. If the amount of masonry can be reduced by arching and the use of buttresses or counterforts, an economic dam may result. In case reinforced concrete slabs are used the reinforcing should be with round rods with nuts and washers on the ends and spliced with sleeve nuts, so as to reduce to a minimum dependence upon adhesion or bond. A good form of reinforcement is a combination of round rods and angles with holes punched in them to receive the rods and act as bearing plates or washers. Rods should have two nuts, one to bear against each side of the metal. Rods should also be used transverse with the main rods of the slabs, so as to tie the concrete together in every direction. At the counterforts or walls supporting the slabs there should be angles through which the rods pass and to which also rods reinforcing the counterforts may connect. This tying together of the entire structure adds greatly to its stability and safety and lessens the probability of extended failure, if a local weakness develops. The deep slabs that would be required in large dams should have a set of rods near the tension side of slab similar to the horizontal rods in beams and a set of rods that curve up about at the quarter points; these are to take the shear and the continuous beam stresses of the slab. All of these rods should pass through angles. The entire system of steel reinforcement should be as rigidly braced and held in place as possible before concrete is placed, so that the placing of concrete will not displace the parts.

The reason that adhesion or bond should not be relied upon in a reinforced concrete dam is because water under pressure lessens the gripping power of the concrete.

A reinforced concrete dam could be made with a vertical slab against the water sustained by counterforts or buttresses on the down stream side.

A dam must be stable against sliding on its foundation. If the foundation of the dam is in rock, notches should be cut in the rock so that slipping will be prevented by the rock itself. In general dependence should not be placed

upon a wall of earth to prevent slipping or a notch in an earth foundation because of the compressibility of earth. Assuming a coefficient of sliding friction of the masonry on the earth of ½, the dam would be stable against sliding if its weight were twice the horizontal pressure of the water. Using the same unit weights as before, it will be seen by a simple calculation that a rectangular dam whose weight is twice the horizontal pressure of the water would have a width of .42 times its height. A triangular dam would have a width or base .83 times its altitude. If water works its way under the dam, it will lubricate the surface upon which sliding takes place. The sliding out of large chunks of masonry in dams that have failed would indicate that this is a mode of failure to be looked for and guarded against. That such a failure may not take place locally it is important that dams be made of monolithic concrete and not of blocks of stone. It is also important that some steel reinforcement be used, even in solid dams, to tie the concrete together. Where in the foundation of a dam the principal bearing surfaces against the earth are made normal to the resultant pressure, sliding is practically eliminated, and settling would cause a uniform movement of the dam of a nature similar to the vertical settlement of a structure. These foundations must be deep enough not to cause upheaval of the earth due to the lateral pressure.

In a dam with counterforts or buttresses inclined shafts could be made use of as foundations for the counterforts or buttresses to take the inclined resultant of the pressure. Being separated from the curtain wall in contact with the water these would not be subject to the buoyancy or the lubricating action of the water under high pressure.

Anchor bolts in the natural rock to tie a dam down against uplift should be used with caution. A bolt dropped in a drilled hole and grouted is not the best kind of anchor. If steel bolts are to be used as anchors, it is better to excavate expanding holes in the rock and fill these with concrete around bolts suspended in them.

Arched dams are sometimes made, which have their *abutments in the* rock on each bank of the stream. Such

dams are, of course, convex on the upstream side. The water acts as a horizontal load on the arch. The curve of the dam in any horizontal section should be the arc of a circle, as this is the shape of a curve of equilibrium for a uniform load normal to the extrados of an arch. The pressure around the arch at any given depth h is 62.5 hR per foot in the height of dam, where $h =$ depth below surface of water and $R =$ radius of arch in feet. The arch could not be a true reinforced concrete arch, because the load is a constant one, and there is no bending moment to resist. It should, however, be tied together with rods runing horizontal and vertical to prevent the concrete from cracking. The unit compression on the concrete (which should be 1:2:4 concrete of Portland cement and good sand and stone) should not exceed 200 or 300 lb. per sq. in., because the concrete is in compression over the entire surface, and there is no reinforcement against diagonal shear. Steel should not be counted upon to take any of the compression of the arch.

The Design of Reinforced Concrete Chimneys.

There are several problems that enter into the design of reinforced concrete chimneys, which are not usually found in text books on mechanics of materials. These will be taken up as a preface to this article.

One of these problems concerns the position of the line of pressure on a hollow rectangular column of elastic material in order to produce only compression on the material. The rectangular column will be taken as one having outside dimensions b and d and inside dimensions b' and d'. A central load P on such a column will produce a unit compression K as per the following equation:

$$K = P \div (bd - b'd') \qquad (1)$$

If the load P be shifted a distance x in a direction parallel to the side d, remaining central in the rectangle in the other direction, it will produce bending in the column in addition to the direct stress. The eccentric load P may be considered as replaced by a central load P and a couple each of whose forces is P, one being located at the shifted position of the load P and the other being at the center of the column but opposite in direction from the other central load, thus neutralizing it. The effect on the column is then a couple which will produce a bending moment Px and a direct central load P. The moment will give tension along one edge of the rectangle and compression along the other edge with uniform variation between. The section modulus of the hollow rectangle is $(bd^3 - b'd'^3) \div 6d$. Hence the extreme fiber stress K' due to the moment Px is

$$K' = (6\ Pxd) \div (bd^3 - b'd'^3) \qquad (2)$$

When $K = K'$, there will be a condition of no stress at one edge of the rectangle and an extreme fiber stress at the opposite edge equal in intensity to double the unit compression due to the direct load P.

For any given rectangle the value of x may be found *by equating* K *and* K' of equations (1) and (2) respec*tively and solving*. This value of x will give the position

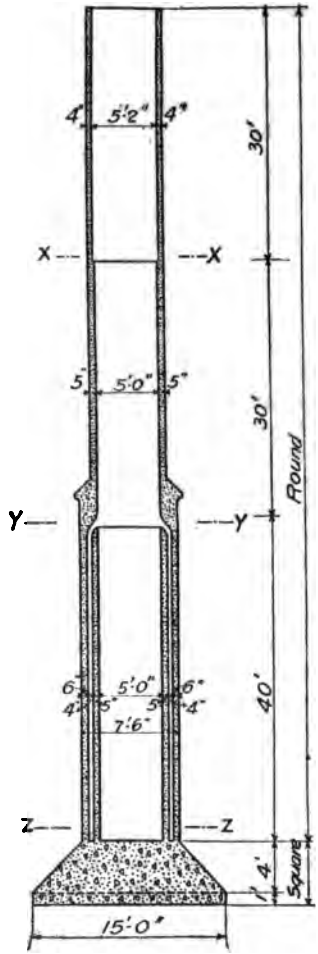

FIG. 2.

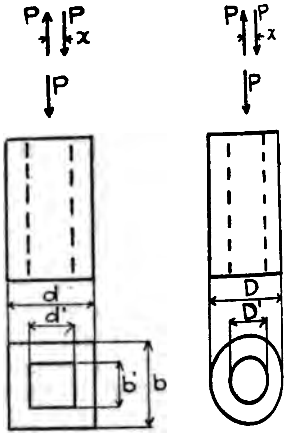

FIG. 1.

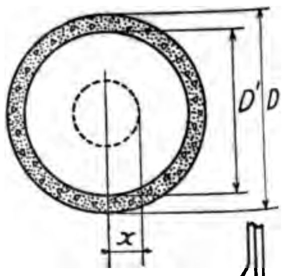

FIG. 3.

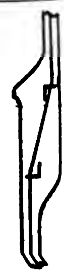

FIG. 4.

of a load that will give the condition named in the last paragraph. A load placed anywhere within a distance x from the center of column will produce only compression in the column. In a hollow square column or chimney whose external and internal diameters are d and d' respectively we find

$$x = (d^2 + d'^2) \div 6d \qquad (3)$$

The resultant pressure on the section of a hollow square chimney should then fall within this distance x of the center.

When $d' = o$, that is, when the section of the column is rectangular, x is one-sixth of d. This is the familiar proposition that the center of pressure in a rectangle must fall within the middle third of the base in order not to produce tension on the extreme fiber. When $d' = d$, $x = 1/3\ d$, showing that when the thickness of the shell is small compared with the diameter the resultant may approach a point one-third of the diameter from the center without producing tension on any part of the shaft.

In a hollow circle whose external and internal diameters are D and D' respectively, we find by a similar operation

$$x = \frac{D^2 + D'^2}{8D} \text{ or } \frac{D}{8}\left(1 + \frac{D'^2}{D^2}\right) \qquad (4)$$

When $D' = o$, $x = \frac{1}{8} D$, which shows that the center of pressure must fall within the middle quarter of the diameter of a solid circular shaft in order not to produce tension on the extreme fiber. When D' approaches equality with D, x approaches $\frac{1}{4}$ of D, showing that in a thin shell the resultant may fall within $\frac{1}{4}$ of the diameter from the center without producing tension on any part of the section.

Another problem that has a bearing on the design of chimneys is that by which the stress may be found in a system of rods disposed in a circle when subject to a bending moment acting on the circle. This will be solved through the medium of a thin tube in bending, which is still another problem useful in the solution of stresses in chimneys. The moment of inertia of a tube or circular shell whose thickness is t, and radius R, both in inches, is

$\pi R^3 t$, and the section modulus is therefore $\pi R^2 t$. If we let $K =$ extreme fiber stress on the tube in pounds per square inch and M the moment on the same in inch-pounds, we have

$$M = K \pi R^2 t \quad (5)$$

(M may be in ft.-lbs., K in pounds per square foot, and R and t in feet.)

Suppose now that the tubular shell is replaced with a system of rods equally spaced and disposed in a circle. Let $n =$ number of rods. Then $2 \pi R \div n$ is the space from rod to rod. The rod that is located at the point of maximum extreme fiber stress can be considered as taking the stress that would come on the shell in an area equal to the spacing of rods by thickness of shell, or $2 \pi R t \div n$. The stress on this rod is then $2 \pi R K t \div n$. Substituting the value of K out of equation (5) in this we have for the stress S on the rod receiving the maximum stress

$$S = 2M \div R n \quad (6)$$

Equation (6) is remarkable. It is derived, as observed, only to be a close approximation for a comparatively large number of rods arranged in a circle. By trial it will be found to be exactly true for 12 rods, 8 rods, 6 rods, 4 rods, and probably for any even number of rods above these numbers. This equation could be used in finding the maximum pull on the anchor bolts of a steel stack. It could be used in finding the stress on the bolts of a pipe coupling under bending. It could be used in finding the stress on the rivets in the circular seam of a riveted pipe under bending, as in the case where the pipe spans an opening.

Another problem that will need to be solved concerns the extreme fiber stress on a hollow circular section under bending. The formula for this case is

$$M = .0982\ (D^4 - D'^4)\ K \div D \quad (7)$$

The calculations on a reinforced concrete chimney or stack involve the determination of the compression on the concrete or the dimensions necessary to keep that compression within certain limits, the embedded steel required to

relieve the concrete of all tension, and the base necessary to give the required stability against the wind.

In the stack shown in Fig. 2 it is desired to investigate the stability and to determine the amount of steel necessary to reinforce the concrete. The areas and weights are found to be as follows:

Area upper section, 829 sq. in. = 5.76 sq. ft.
Area middle section, 1021 sq. in. = 7.09 sq. ft.
Area outer shell, lower section, 1583 sq. in. = 11.00 sq. ft.
Area inner shell, lower section, 1021 sq. in. = 7.09 sq. ft.
Weights at 150 lb. per cu. ft.:

Upper section	25,900
Middle section	31,900
Outer shell, lower section	66,000
Inner shell, lower section	42,500
Plinth, base	33,700
Frustum, base	78,800
Total weight of stack	278,800

The moment of stability of the stack is 278,800 times one-sixth of the width of the base, or 697,000 ft.-lb. (The added weight due to the thickening up at section *Y Y* is omitted here to simplify the calculations.)

The wind loads (at 40 lb. per sq. ft. on one-half the projection of the cylinder) are as follows:

Upper section, $40 \times \frac{1}{2} \times 5.83 \times 30 = 3{,}500$ lb.
Middle section, $40 \times \frac{1}{2} \times 5.83 \times 30 = 3{,}500$ lb.
Lower section, $40 \times \frac{1}{2} \times 7\frac{1}{2} \times 40 = 6{,}000.$ lb.

The load on the upper section is applied 85 ft. above the ground, that on the middle section is applied 55 ft. above the ground, and that on the lower section is applied 20 ft. above the ground. The wind moments at the ground level are

$$3{,}500 \times 85 = 297{,}500$$
$$3{,}500 \times 55 = 192{,}500$$
$$6{,}000 \times 20 = 120{,}000$$

Total.....610,000 ft.-lb.

At the base of the foundation there will be added to this

moment $13{,}000 \times 5 = 65{,}000$, making the total 675,000 ft.-lb. This is less than the moment of stability previously found. Hence the stack is stable against the wind.

The area of the base is 225 sq. ft. Dividing this into the total weight found above we find a pressure on the soil of 1,240 lb. per sq. ft. When the wind load is applied, the pressure on the soil on the leeward edge is almost double this amount. (It would be just doubled if the overturning moment just equaled the moment of stability.) A pressure of 2,400 lb. per sq. ft. is very low for ordinary soil.

At the section XX the weight of the stack is 25,900 lb. and the compression due to dead weight is 31 lb. per sq. in. The wind moment at this section is $3{,}500 \times 15 = 52{,}500$ ft.-lb. The concrete is able to resist a certain part of this without suffering any tension on the extreme fiber. The distance from the center that the resultant pressure may fall to give this condition of no tension is the value of x in equation (4). Solving for the section XX we find x is 1.30 ft. Hence (taking moments about a point x feet from the center of stack) the moment of stability of the stack, without relying upon the steel, is $25{,}900 \times 1.30 = 33{,}700$ ft.-lb. This leaves 18,800 ft.-lb. to be taken by the steel.

At section YY the wind moment is $3{,}500 \times 45 + 3{,}500 \times 15 = 210{,}000$ ft.-lb. The value of x is 1.27 ft., and the moment of stability of the concrete is $57{,}800 \times 1.27 = 73{,}400$ ft.-lb. This leaves 136,600 ft.-lb. to be taken by the steel.

At section ZZ the wind moment, previously found, is 610,000 ft.-lb. The value of x is 1.64 ft. and the moment of stability of the concrete is $123{,}800 \times 1.64 = 203{,}000$ ft.-lb. This leaves 407,000 ft.-lb. to be taken by the steel.

For the steel reinforcement we will take the neutral axis in the center of the shaft in all cases and allow the concrete to take whatever compression develops. The critical point is then the outermost rod on the tension side. *The tension* on this rod is found by equation (6). Or *assuming* a size of rod the number of rods required may be *'d by* the same equation.

Assuming that ⅞-in. round rods will be used vertically. At 12,500 lb. per sq. in. the permissible tension on one rod is 7,520 lb. Keeping these rods 3 in. from the outer surface of the stack, the radius of their circle is 3.5 ft. Substituting in equation (6) we have $7{,}520 = 2 \times 407{,}000 \div 3.5\, n$, from which $n = 31$ bars.

At section $Y\,Y$, using the same bars and 2 ft. 9 in. for the radius, we have $7{,}520 = 2 \times 136{,}600 \div 2.75\ n$, from which $n = 13$.

If 32 bars be used in the lower section, half of them may be dropped at section $Y\,Y$.

At section $X\,X$ but little steel is required. There should be some rods, however, to prevent the cracking of the concrete. Eight of the ⅞-in. rods could be continued to the top of chimney, or smaller rods could be used, say 16 half-inch round rods lapping those of the section below. Another method would be to run four of the ⅞-in. rods the full length and to use smaller rods between these to tie the concrete together.

The ⅞-in. rods should have anchorage in the foundation; preferably by a circular angle punched to receive them, or by anchor plates. Two nuts on the end of each rod would insure a bearing on the metal. Where rods join, they should be threaded and have sleeve-nut splices.

In the horizontal direction rods in circles should be placed about every 20 or 30 in., their ends being lapped a foot or more. These rods could be ½ in. square. They should be wired to the vertical rods.

Around openings in the side additional rods are needed for reinforcement, also possibly a thickening of the concrete.

Where the diameter of the outer shell changes there is a weak section. The bend in the rods induces horizontal thrust, and there is shear on the concrete because of the change in direction of the line of pressure. It is recommended that the bends in rods be made in the thickened portion of the shell and that curved angles be used at these bends to stiffen the concrete, as shown in Fig. 4.

The inner shell needs rods both vertically and horizontally

to keep the concrete from cracking. This inner shell is kept separate from the outer shell, except near the bottom, where the air space is sometimes filled in. The purpose is to allow free expansion and contraction of the highly heated part. The amount of steel used to reinforce concrete where there is no calculable stress is usually made from 1-500 to 1-1000 of the area of concrete.

The compressive stress on the concrete is another matter for investigation. *Z Z* is the critical section; other sections will be found to have considerable less unit stress. The compression from dead load is 78 lb. per sq. in. By equation (7), using 610,000 for *M*, we find an extreme fiber stress of 33,800 lb. per sq. ft. or 235 lb. per sq. in. This makes the maximum compression at this section 313 lb. per sq. in., which is about right. Concrete in such case should not be subject to the usual allowable compressive unit of 500 lb., because it is not confined on one side, as in the case of beams and slabs. This concrete is more in the nature of a column. The hoop reinforcement does not have the efficiency in this case that it has in a hooped column. As a check on the compressive stress on the concrete we may use the same moment in equation (5) and the mean value of *R*, or 3.5. The result is 220 lb. per sq. in. as compared with 235 lb. found by the more exact method.

The design of the chimney would be simplified and improved, from a structural standpoint, if the offset in the outer shell could be avoided.

For the outer shell of this chimney the concrete used should be a 1:2:4 mixture with small sized broken stone of good quality. The inner shell should be a concrete that will resist a moderately high heat. No limestone should be used in this part. Small gravel and sand or trap and sand would be suitable.

If sand alone is used with the cement such mixtures as 1:7 or 8 should not be employed. The voids in sand are too great to be filled with this small proportion of cement. About 4 or 5 parts of sand to one of cement ought to be

e limiting ratio, and even this ratio should only be used ith a very coarse sand.

The spread of the footing of this chimney is not enough require much if any reinforcement in the concrete. It ould, however, be desirable to have some rods laid parllel to each side and some laid diagonally in both dictions.

shell of very long radius, the stresse
roof are capable of more exact det
analysis of these stresses is compar

The stresses in a conical covering on a circular tank will first be consid covering such as that shown in Fi above the line AB be represented by solved into two components, namel cone not capable of taking any be pression and tension may resist both of forces. (The cone is somewhat tube under external pressure so th relatively small it may be in a st librium.) The component T will gi direction of an element of the con H, being continuous around a circl acts like external pressure on a cyli pression also.

The total amount of T is found by

$$T=\frac{Wmr}{h}$$

The area upon which this force
radius is r and whose thickness is t

The force H in Fig. 1 is resisted by hoop compression in horizontal planes. The force T is cumulative and reaches the value in equation (1) only at the foot of the slope mr. The force H is resisted by rings or hoops of stress distributed along the entire surface of the cone, that is; these rings make up the cone. In order to arrive at the amount of this hoop compression we observe that each ring is in effect an element of a cylinder under external pressure. It is well known that the tension or compression on the shell of a cylinder due to a radial pressure, outward or inward, of any given intensity is equal to the amount of that pressure on a radius. In other words the tension, or compression on the ring or hoop is equal to the total pressure around the circumference divided by 2π. Therefore H divided by 2π is equal to the stress on an element mr of the cone. But $H = Wr \div h$, and the total compression along the element mr is

$$K=\frac{Wr}{2\pi h} \tag{3}$$

This compression is not uniformly distributed along the element, but varies in intensity. In order to find the value of K, for the case where the weight is only that of the cone itself, we will substitute for W the weight of the cone above the section AB, or $w\ m\ \pi\ r^2\ t$, where $w =$ weight per cu. ft. of the material of the shell other dimensions being in feet. Then

$$K=\frac{wmr^3t}{2h} \tag{4}$$

The unit compression at any section is found by substituting in (3) an element of the weight of the cone and dividing by an element of the area. Letting P represent this unit compression we find

$$P=\frac{wr^2}{h} \tag{5}$$

If in equation (2) we substitute for W the weight of a conical shell whose radius is r, or $\pi\ r^2\ m\ w\ t$, we shall have

$$C=\frac{wm^2r^2}{2h} \tag{6}$$

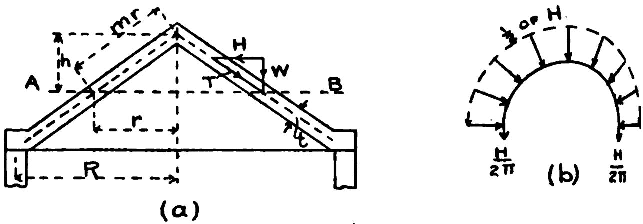
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Fig. 1.

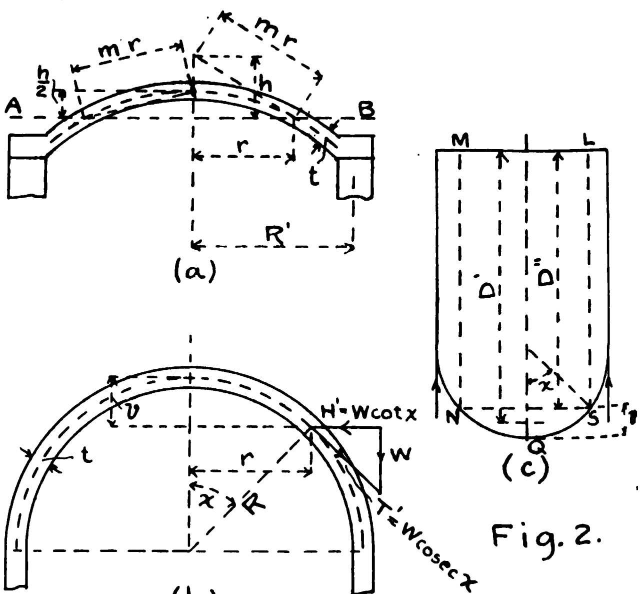
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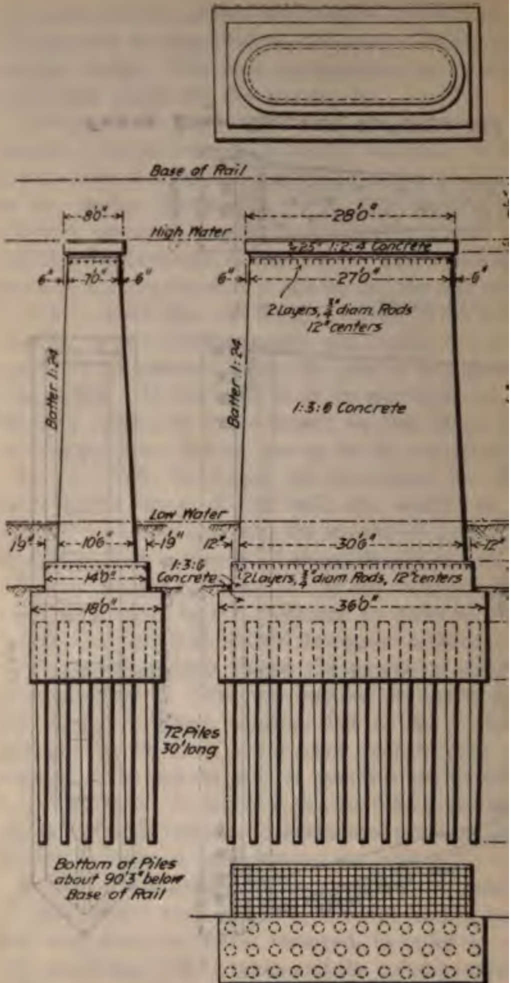
Fig. 2.

We see from equations (5) and (6) that the thickness of the conical shell does not enter in the determination of the unit stress in a cone supporting its own weight only (in a shell where the volume is sensibly equal to the area of the median cone by the thickness). It may also be shown by equating C and P in equations (5) and (6) that when m equals the square root of 2, the intensity of stress in both directions is the same. This is true when the angle of the element of the cone with the horizontal is 45 deg.

The thickness of concrete in the cone is determined by other considerations than the weight of the shell itself. A uniform load, such as a snow load uniformly distributed would not add much stress. The weight of a man on the roof of a tank or snow load on one-half only are possibilities that must be considered. These scarcely admit of calculation. Hence a simple cone without ribs would be designed largely by judgment. A thickness of shell of 4 in. for a tank 15 ft. in diameter and 6 in. for a tank 25 ft. in diameter would probably meet the requirements. Steel rods in circles around the cone and rods down the slope would be valuable to insure the integrity of the roof and to guard against shrinkage cracks. If any man holes are to be left in the roof, the concrete around the openings should be reinforced with steel rods. The circular and sloping rods in the conical covering are not true reinforcing rods, because they would not take tension. There is no basis for finding their size for the same reason.

At the foot of the slope, if the cone is supported on vertical walls, there is a tension which is equal to the total compression exerted on a vertical section from the apex to the foot of the slope, that is, the sum of all of the hoop compression in the horizontal rings. The amount of this is found by equation (3) by substituting R for r. For example, given a tank 15 ft. in diameter, with $m = 1\frac{1}{4}$, the concrete being 4 in. thick, and $h = 5.63$ ft. The weight of the concrete in the conical roof is 11,000 lbs., at 150 lbs. per cu. ft. A snow load at 30 lbs. per horizontal square foot would weigh 5,300 lbs. Using 16,300 for W



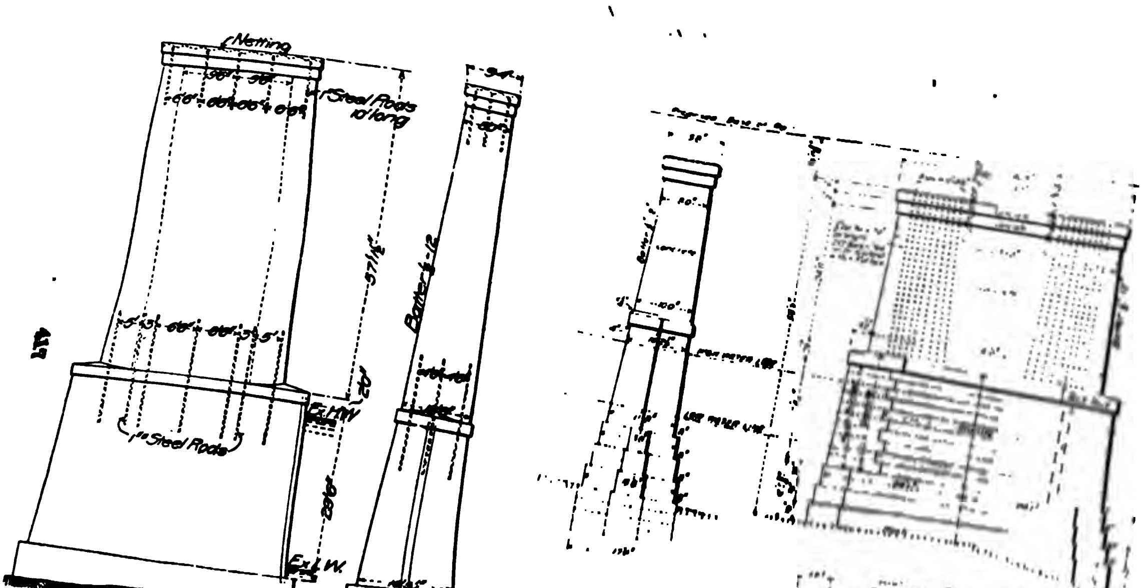
Plan and Elevation of River Piers.

weight of bottom of tank and liquid contents. This must be resisted by concrete, and there should be area enough of concrete close to this corner to take the compression at about 200 lbs. per sq. in. It is of course uncertain just how much of the walls and bottom may be included in the ring at the junction to take this compression. A fair proportion would include twice the thickness of wall up the side and twice the thickness of bottom down the slope. Then the concrete over the supporting wall should be thickened as shown in Fig. 1.

A dome may be analyzed in a manner somewhat similar to a conical covering. In the dome, as in the conical roof, a thin shell will support uniform load by tensile and compressive stresses alone, though if the thickness is relatively small the equilibrium will be unstable.

If the dome is segmental, as at (a) Fig. 2, and parabolic in section, a portion having a base whose radius is r will have stresses at the foot of the slope (in the direction of the slope) corresponding to those in a cone whose altitude is h, the cone being tangent to the dome. Because of the fact that it is a property of a parabola the altitude of the cone will be twice that of the dome, as indicated. It is to be remembered that n is not a constant in this case, as in the case of the cone. For the weight of the dome itself the superficial area is very close to the area of a circle whose radius is the chord $m'r$. (The area of a segment of a spherical surface is exactly equal to the area of a circle whose radius is this chord of one-half of the arc.) The volume is then equal to the product of this area and the thickness t. For a segmental dome, as shown at (a), reinforcement is needed over the supporting wall in the shape of a steel hoop to take the thrust. Assuming a dome 20 ft. in diameter and 5 in. thick, we find a weight of 22,800 lbs. and a snow load of 9,400 lbs. If the dome has a rise of 4 ft. at the center, $h = 8$. Substituting in eq. (3) we find $K = 6{,}400$ lbs. At 10,000 lbs. per sq. in. this would require a $\frac{1}{1}\frac{5}{8}$-in. round rod.

The circular stresses (at other sections than directly



Concrete Pier of New River Bridge.

The unit intensity of this hoop compression is found by differentiating the value of K' in eq. (11) and dividing by $R\ t\ dx$, which is the differential increment to the area. The result is a unit compression P' as per the following equation.

$$P' = Rw\left(\cos x - \frac{1}{\cos x + 1}\right) \qquad (12)$$

By examination of equation (12) it will be seen that for $x = o$ the intensity of stress is $\frac{1}{2}\ R\ w$, which is the same as that of the thrust, found by equation (9), at the crown of dome. As this is positive the stress is compression. When $x = 90$ deg., $P' = (-\ Rw)$, which is also the same in intensity as the thrust found by equation (9), over the supporting wall. As this stress is negative it is tension. When $\cos x = \frac{1}{2}\ (\sqrt{5} - 1)$, $P' = o$. This is true when $x = 51$ deg.-49 min. There is thus a point where the intensity of the hoop stress is zero. Above this point the hoop stresses are compression and below it they are tension. Where compressive stresses occur, no steel reinforcement would be demanded for these stresses alone. Where tensile stresses occur, steel reinforcement would be required in a concrete dome. The tension on steel reinforcement would be found by solving eq. (12) for the unit stress at any point and multiplying this by the area of concrete surrounding a rod. In a dome 30 ft. in diameter and 6 in. thick at a section close to the supporting wall the unit stress $= R\ w$ or $15 \times 150 = 2{,}250$ lbs. If we assume rods one foot apart, one rod would be surrounded by $\frac{1}{2}$ sq. ft. of concrete and have a stress of only 1,125 lbs.

There is no thrust to take at the springing of this dome, as in the cone or segmental dome, as the sides start vertical. In the cone and segmental dome there is a hoop stress at the base or springing which just balances the compressive hoop stresses along the vertical section through the axis. In the hemispherical dome this equality or balancing of horizontal forces is accomplished, as explained under equation (12), by positive and negative hoop stresses along the meridian.

In a segmental or hemispherical tank bottom the

stresses for the weight of the bottom alone will be equal in intensity to those found for the dome but reversed in sign. The unit stress in the direction of the meridian is found by the following equation.

$$C_1 = \frac{w' D' R}{2t} \tag{13}$$

where C_1 = unit tension at point S, Fig. 2 at (c), w' = wt. per cu. ft. of liquid, D' = average depth of liquid in $M L S Q N$.

The unit stress in horizontal rings may be found by the following equation.

$$P_1 = \frac{w' R}{t}\left(D'' - \frac{D'}{2}\right) \tag{14}$$

where P_1 = unit tension at point S, and D'' = depth of liquid to point S.

The depth D' is found by adding to the altitude of the cylinder $M N S L$ the average altitude of the spherical segment $N Q S$. The average altitude of the segment of a sphere lies between 2/3 and ½ of the altitude; that is, the volume will lie between 2/3 and ½ of the volume of a cylinder of the same base and altitude. When the angle x approaches zero, the coefficient is one-half, which is the exact coefficient for a paraboloid, and a flat circular arc approaches coincidence with a parabola. When $x = 30$ deg., the coefficient is .512; when $x = 45$ deg., it is .529; when $x = 60$ deg., it is .556. For $x = 90$ deg., or when the segment is a complete hemisphere, the coefficient is 2/3.

To reduce tension in the concrete, as found by equations (13) and (14) to tension in steel reinforcing rods, multiply the unit values there found by the assumed spacing of rods times the thickness of concrete.

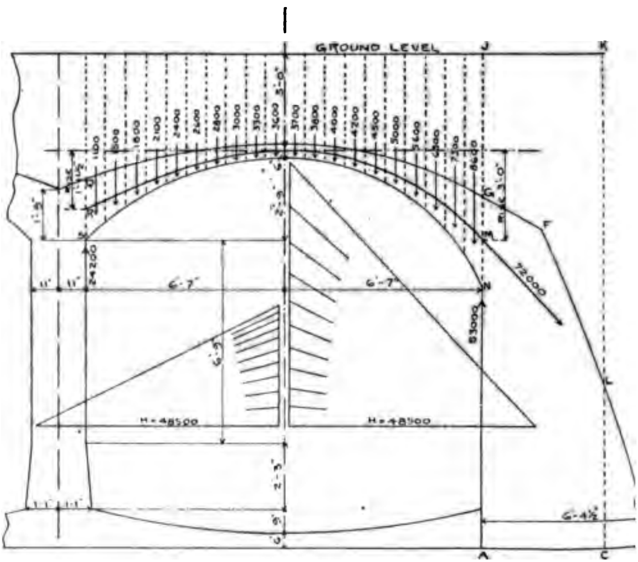
In a ribbed dome, that is a dome supported by a system of arched ribs meeting in a circle or regular polygon at the top, each rib is like the half of an arch. Between the ribs the covering may be a slab of reinforced concrete. This slab would be designed according to the methods of design used for floor slabs. As the inclination of the slab to the horizontal increases, the possibility of carrying a

snow load decreases, and the effect of the weight of the slab itself in producing bending diminishes. The component of the dead weight that produces bending in the slab is the weight times the cosine of the angle made with the horizontal. For the stress in the rib lay a rib out to scale and estimate the weight carried at points at intervals along its length. Take moments of these several loads and the reaction at the foot of rib around the upper end of the rib. This moment divided by the vertical distance from upper end to foot of rib will give the horizontal thrust. From these forces construct the equilibrium polygon. (See similar operation and construction farther on in the case of filter vaults.) This equilibrium polygon will give the direct stress in the rib and will give the bending moment at any section. For the latter, where the equilibrium polygon departs from the center line of the rib, the bending moment is equal to the product of the force in the polygon and the distance from the center of the rib to the force, measured perpendicular to the force.

A form of vault much used for filter covers consists of a system of groined arches carried by square posts or piers, spaced equal distances in both directions. These arches spring in four directions from the piers. At the piers the arches are of course the width of the pier, and at this section they are thickened vertically. Midway between the piers the width of concrete arch is equal to the distance center to center of piers. The concrete is usually plain, and the vaults are covered with three feet or so of earth. Some such vaults are described in *Engineering News*, November 8, 1906, and *Engineering Record*, May 19, December 15, December 22, 1906, April 27, 1907.

The load to be carried consists principally of the dead weight. Allowance should also be made for the superimposed load of teams driving over the ground. In this investigation only the dead load will be calculated. The live load could be considered as a uniform load of 100 or 200 lbs. per sq. ft., since the deep earth fill serves to distribute well the live load.

Fig. 3 shows a section of a filter vault taken through


GROUND LEVEL
J
K
F
L
N
A
C
72000
53000
24200
6'-7"
6'-7"
11"
11"
H=48500
H=48500
2'-3"
6'-4½"
Fig.3

a pier and the outside wall or abutment. This illustrates the various points to be considered, namely, the pier, the half of one of the groined arches, the half arch springing from the outer wall, and the outer wall or abutment. The distance c. to c. of piers is 15 ft. The span of arch is the distance from face to face of piers or from face of pier to inner face of outer wall; this is 13 ft. 2 in. Half of this is divided into 10 equal parts, and the weights of concrete and earth in each part are calculated. These weights are shown in Fig. 3; those on the left side of the center are for the half of a groined arch, and those on the right side are for the half barrel arch.

By taking moments around the center of arch at crown, using the applied loads to the left and the reaction 24,200 lbs., and dividing the moment thus found by the rise, 1 ft. 11½ in., we obtain the horizontal thrust, H, or 48,500 lbs. The rise of the arch is the vertical distance from the center line of arch ring at springing (center point of line QS) and the center of arch at crown. With this thrust H and the several vertical loads the stress diagram to the left is drawn. By drawing lines parallel to these rays the equilibrium polygon beginning at R is constructed for the left half. By calculating the horizontal thrust from the bending moment at center of arch it is assured that the equilibrium polygon will pass through the center line of the arch ring at the crown. It is seen that this polygon lies within the middle third of the arch ring throughout. Hence the arch will be stable, if the unit stress is not too great.

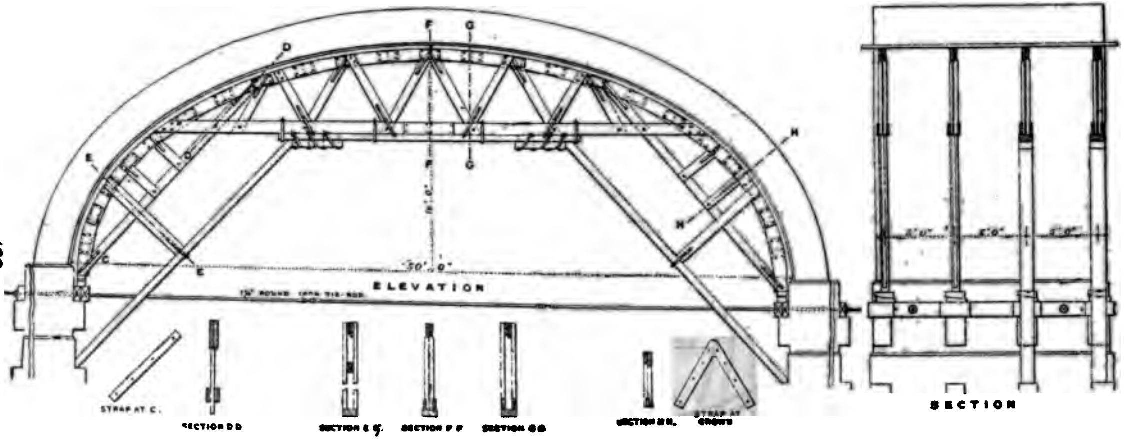
The thrust for the half arch to the right must be the same as that to the left, but, as the load is more, the rise must be greater. The rise in this case was found by first laying out a trial arch and estimating the weight and finding the moment. Dividing this moment by the known thrust gave the rise to agree with the calculated loads. By another trial a rise and moment were found that agreed with the thrust. The equilibrium polygon for this half arch is found as for the other. It also falls inside of the middle third of the arch. In all cases the curves for top

and bottom of the arches were made arcs of circles. It will be seen that the equilibrium polygons almost coincide with arcs of circles. The unit compression on the arch is seen to be only about 100 lbs. per sq. in.

The thrust of the several arches against the piers must balance. No part can be taken by the piers. If the arches are not loaded alike, the additional thrust must be absorbed in the arches themselves. For this there is a surplus strength imparted by the horizontal stiffness. The pier will then be in simple compression. The load is about 100,-000 lbs., and, as the area is 484 sq. in., the unit compression is a little more than 200 lbs. per sq. in. This is not excessive for a pier of this length, not reinforced, if made of good 1:2:4 concrete, when the pier is not subject to any side forces. If live load is to be provided for, however, the pier should be made larger, so that there will be no more than about 200 lbs. per sq. in. in compression.

On the wall the forces are the 72,000 lbs. due to the arch, acting on 15 ft. of wall, the weight of 15 ft. of *ABDEFG* in concrete, and the weight of 15 ft. of *JGFLK* in earth. The moments of these should balance about *C*, which is 1-3 of *AB* from *B*. If the moment of the concrete and earth is not sufficient to balance that of the 72,000 lbs., the wall should be made wider at the base. With the dimensions shown the moments were found to balance about *C*. Only the part of the earth load to the left of *CK* is taken, because that to the right does not increase the stability of the wall (considering its weight alone). It cannot be said to decrease the stability, hence it is neglected. If water is in the filter, not balanced by saturated earth outside, its horizontal pressure should also be considered as a force against the wall.

In the construction of these vaults the forms should be kept under two or three rows of vaults beyond any that may have the forms removed, in order to make sure that the thrust will be taken and the columns relieved from taking any of it.



Details of Falsework for the Elliptical Concrete Arches of the Cannington Brid

From Eng. Record, Vol. 52, p. 472

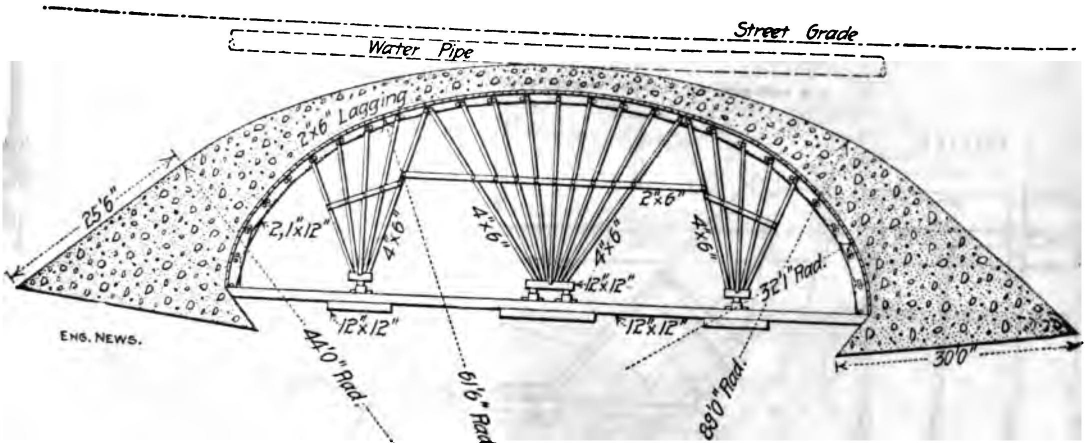
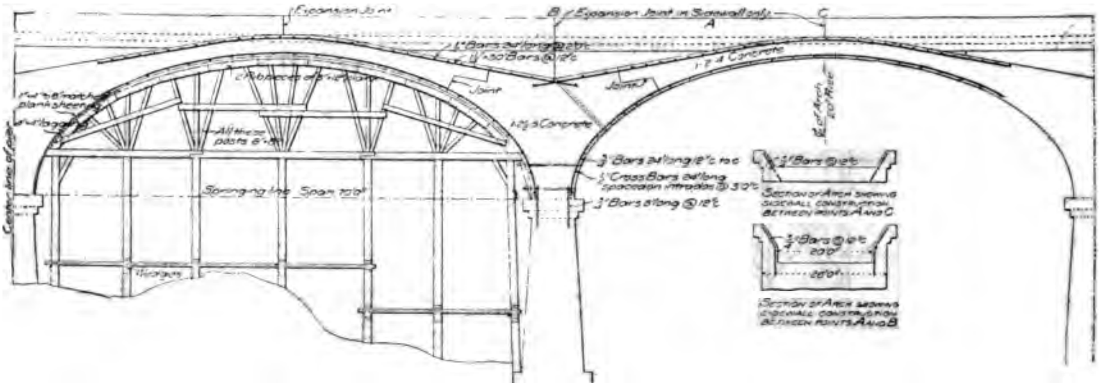


FIG. 2. CENTER FOR SAN LEANDRO CONCRETE ARCH BRIDGE.

From Eng. News, Vol. 50, p 74



Cross Section of Two Spans of Reinforced-Concrete Arch Bridge on Southern Ry.

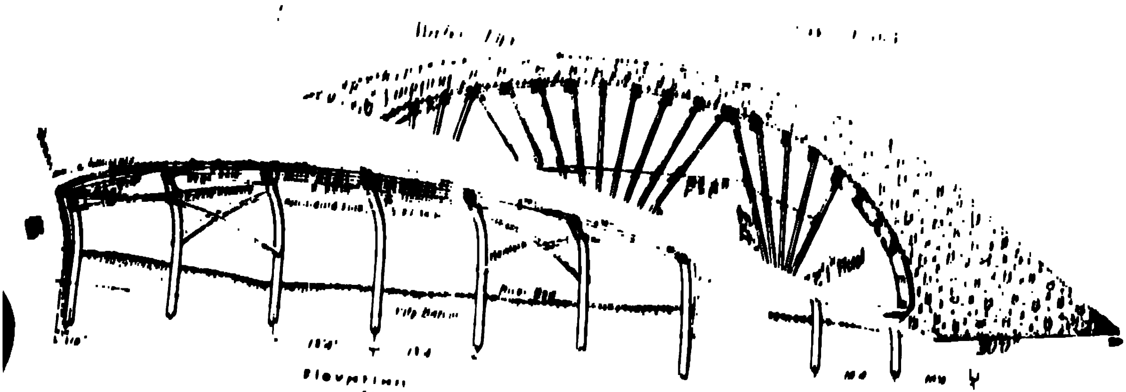
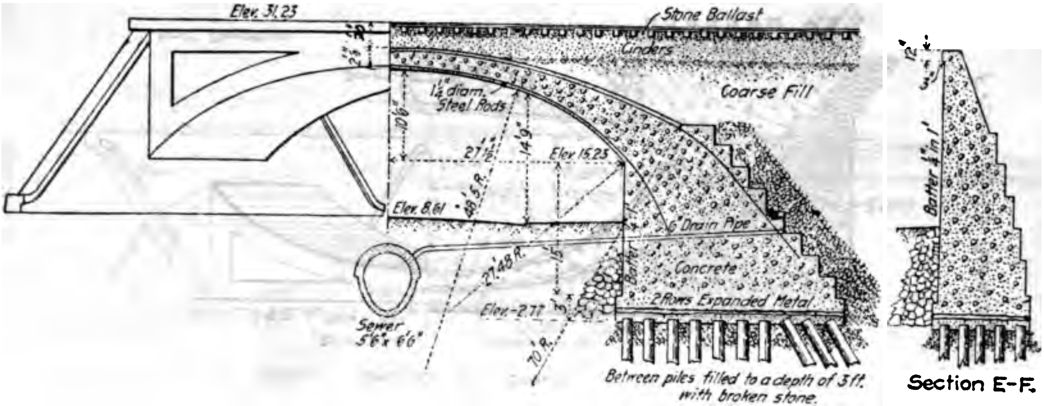


FIG. 3. CENTER FOR 88 FT. SPAN, GRAND RAPIDS BRIDGE.

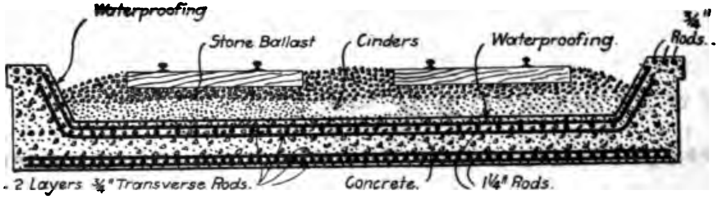
From Eng. News, Vol. 38, p. 490



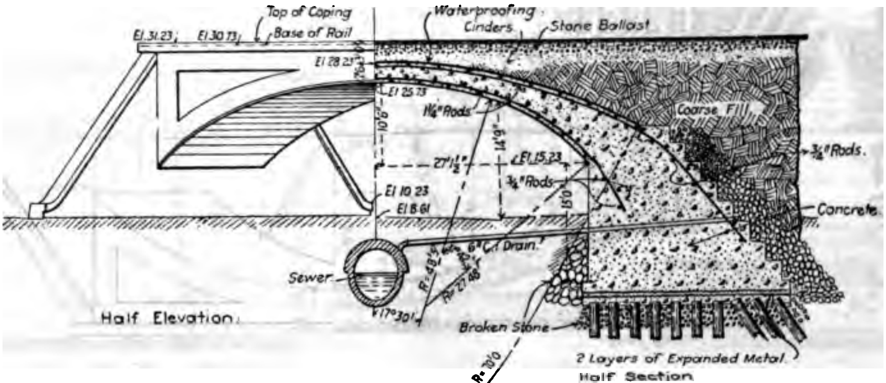
A Double Track Concrete Steel Arch, Central Railroad of New Jersey.

From R. R. Gaz. Vol. 37, p. 311

From Eng. News, Vol. 50, p.168



Section at. Crown.



Reinforced Concrete Railway Bridge at Jackson Street, Newark.

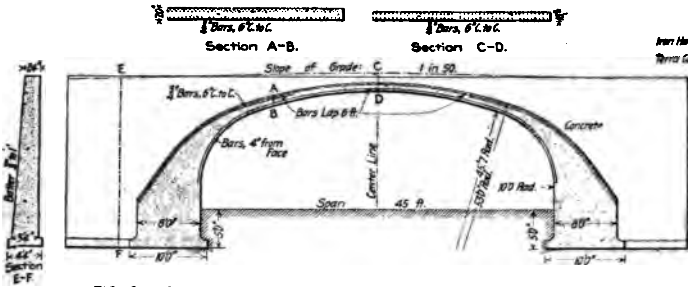


FIG. 2. LONGITUDINAL SECTION OF FOREST PARK CONCRETE-STEEL BRIDGE.

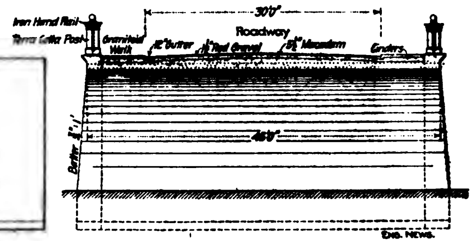


FIG. 3. TRANSVERSE SECTION OF FOREST PARK BRIDGE.

From Eng. News, Vol. 49, p. 531

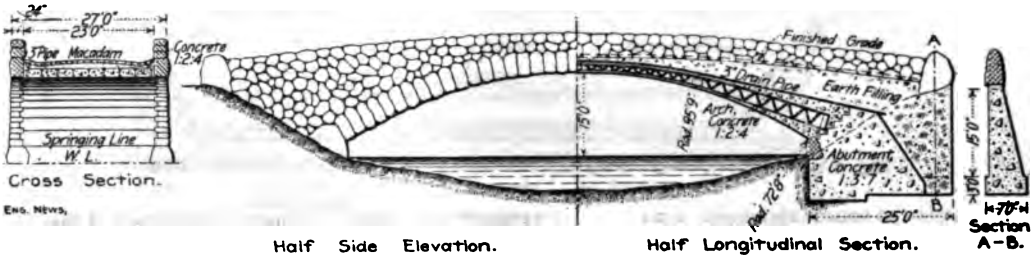


FIG. 2. ELEVATION AND SECTIONS SHOWING CONSTRUCTION OF BOULDER-FACED CONCRETE-STEEL BRIDGE.

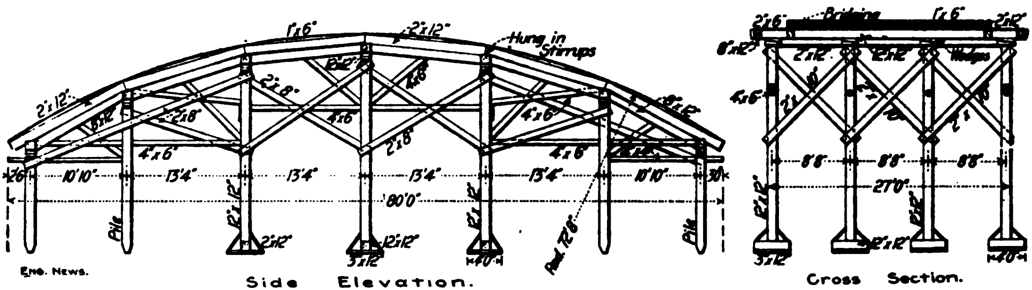
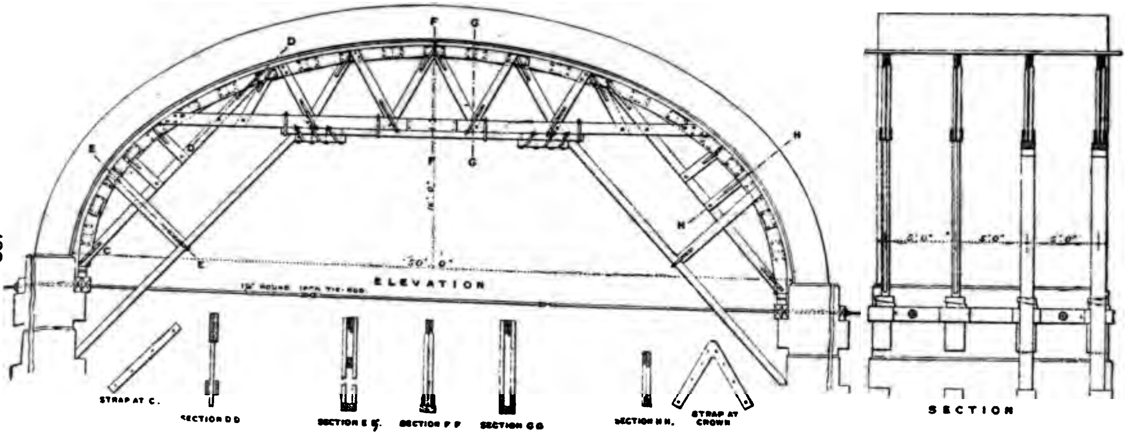


FIG. 3. TIMBER CENTER FOR BOULDER-FACED CONCRETE-STEEL BRIDGE.

From Eng. News, Vol. 48, p. 110



Details of Falsework for the Elliptical Concrete Arches of the Cannington Brid

From Eng. Record, Vol. 52, p. 472

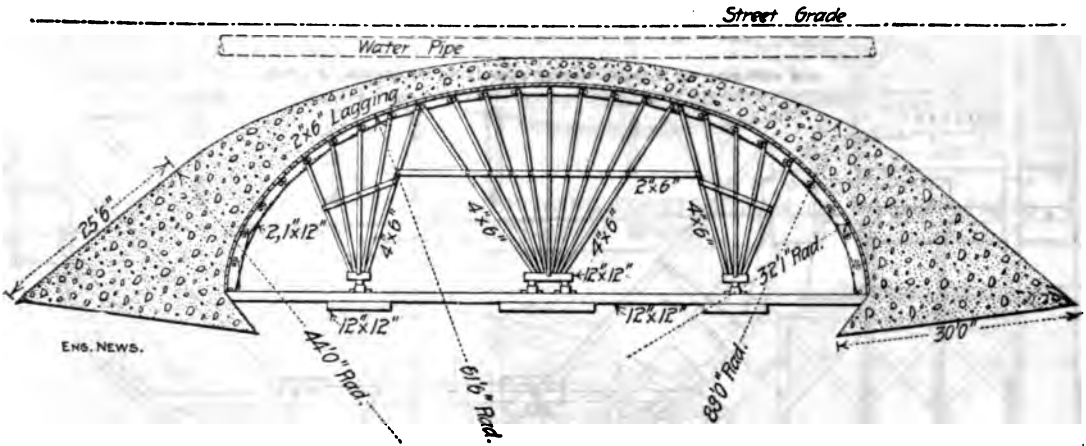
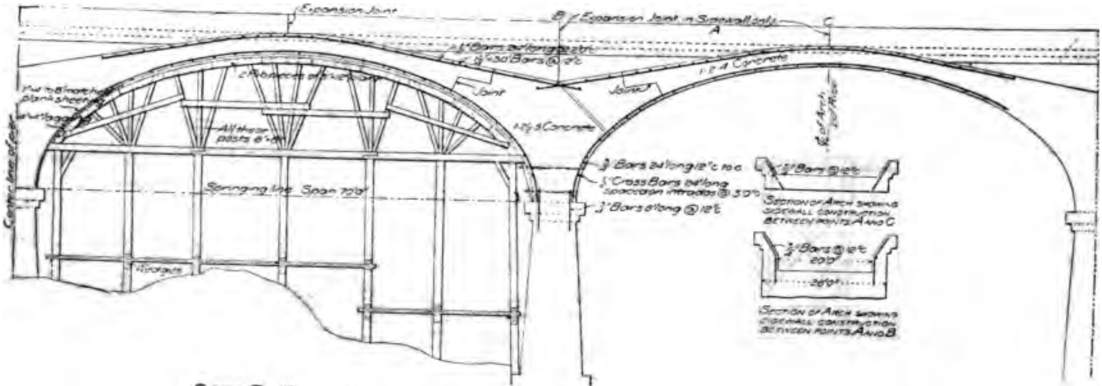


FIG. 2. CENTER FOR SAN LEANDRO CONCRETE ARCH BRIDGE.

From Eng. News, Vol. 50, p 74



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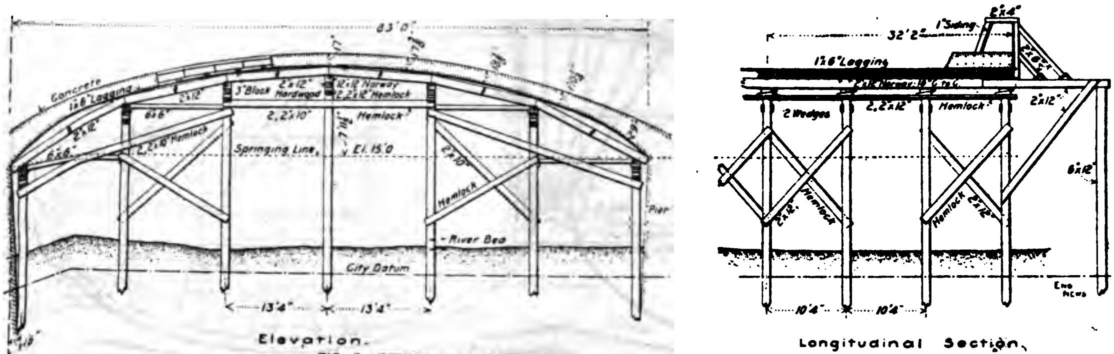
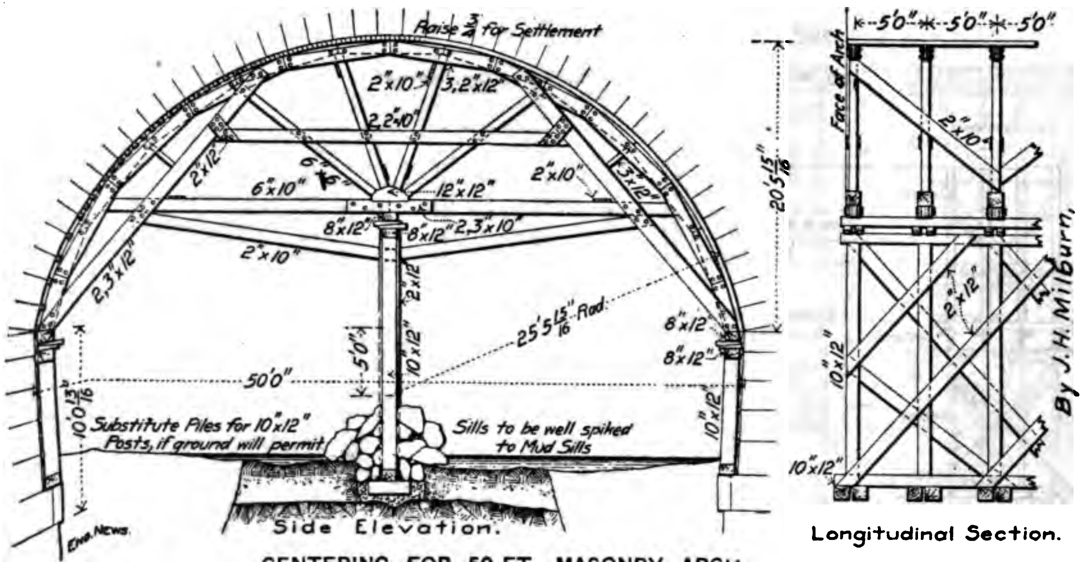


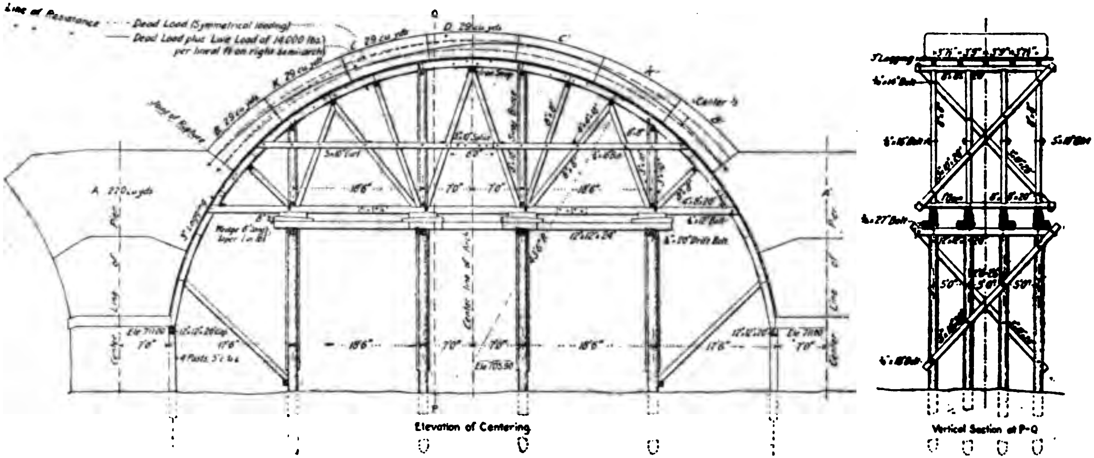
FIG. 3. CENTER FOR 83-FT. SPAN, GRAND RAPIDS BRIDGE.

From Eng. News, Vol. 52, p. 490



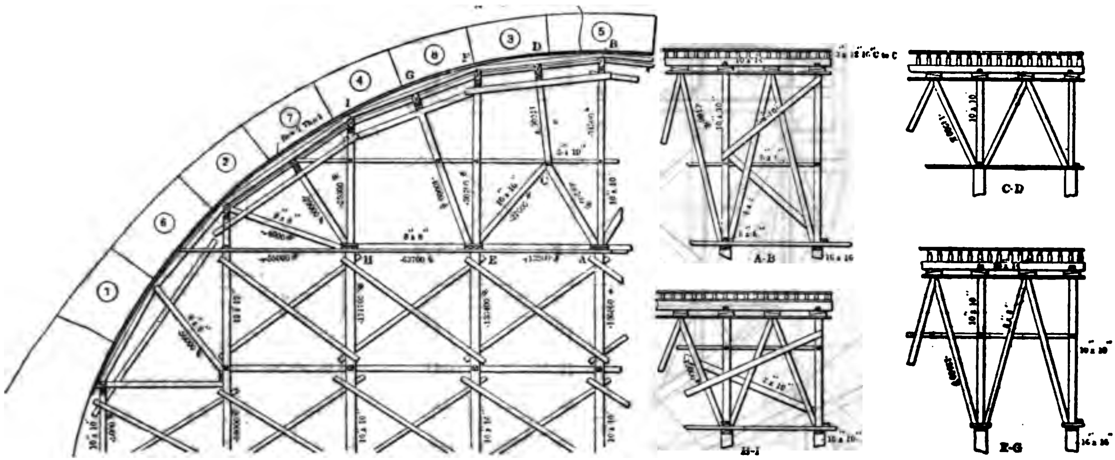
CENTERING FOR 50-FT. MASONRY ARCH.

By J. H. Milburn,
From Eng. News, Vol. 56, p. 207



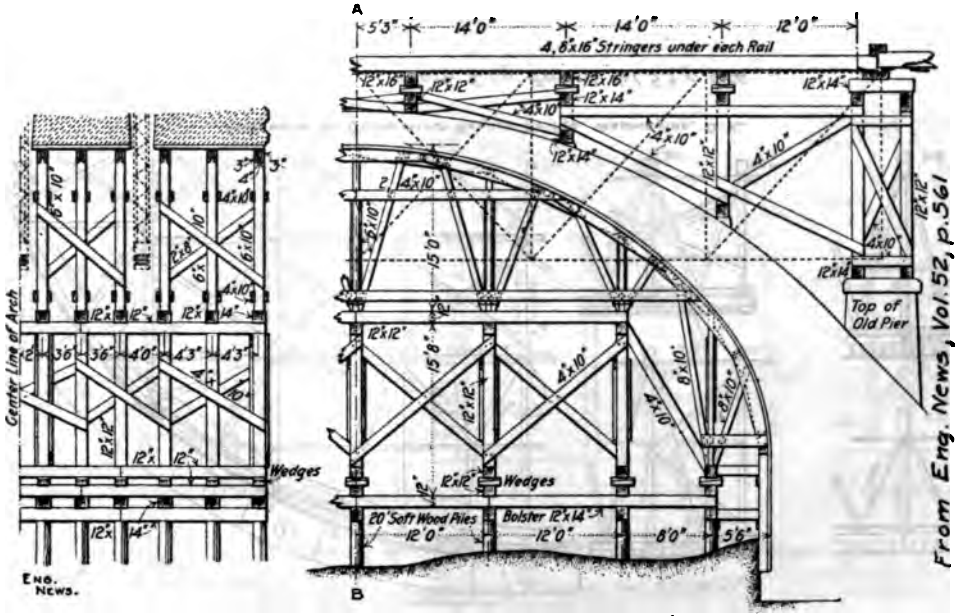
Centering Used on the Santa Ana Viaduct at. Riverside, Cal.

From Eng. Record, Vol. 52, p. 286



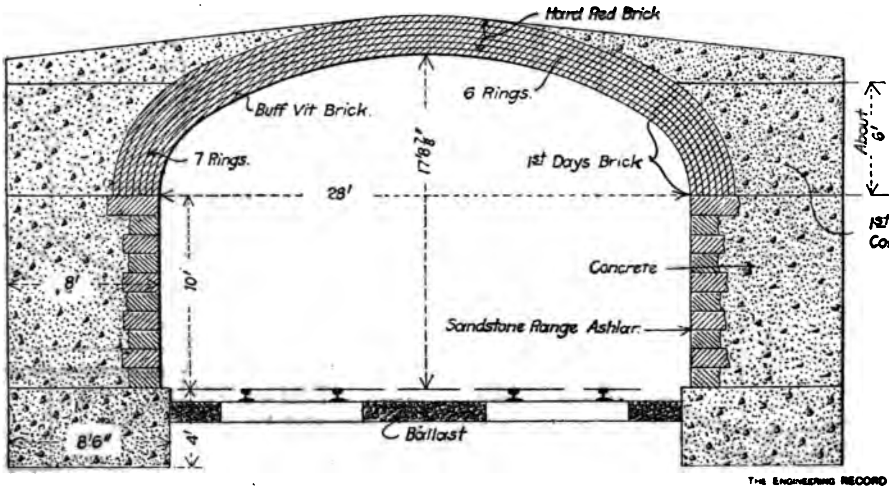
Falsework for Connecticut Avenue Bridge, Washington, D. C.

From Eng. Record, Vol. 53, p. 676



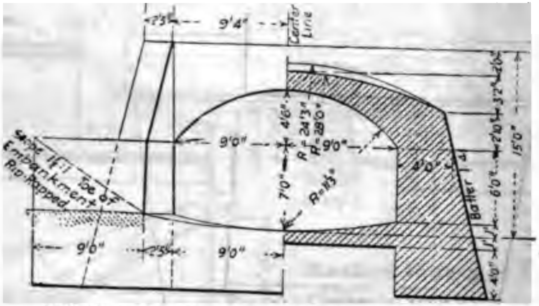
Section A-B. Part Elevation.

FIG. 4. DETAILS OF CENTERING FOR CONCRETE ARCH AT PLANO, ILL.

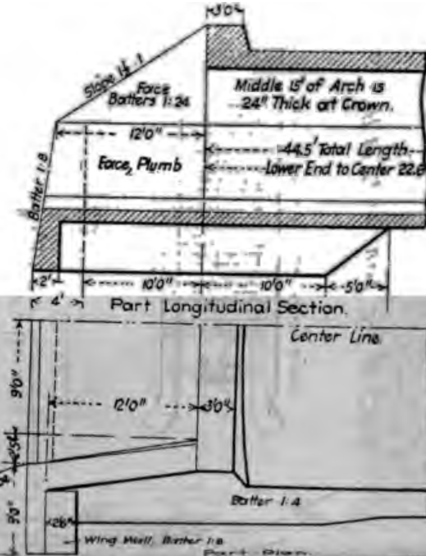


Typical Section of Virginia Avenue Tunnel.

From Eng. Record, Vol. 50, p25

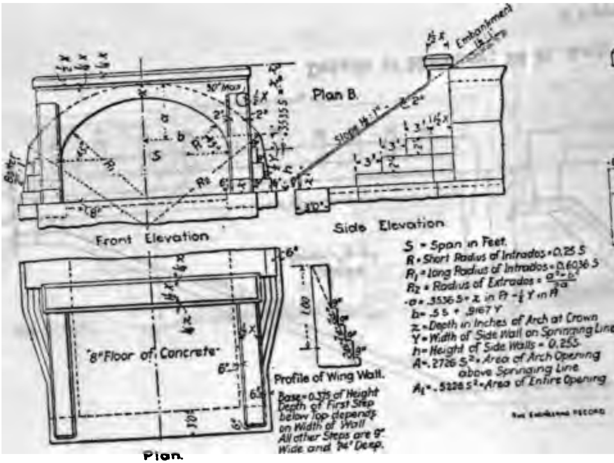


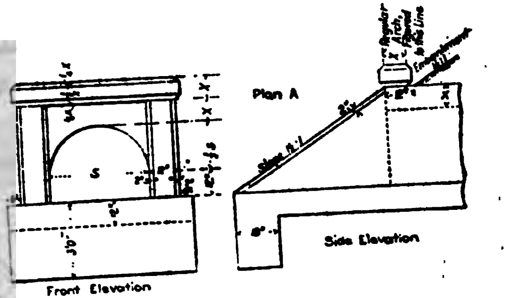
Half End Elevation. Half Transverse Section



Eighteen-foot Culvert Details,
Bangor & Aroostook R. R.

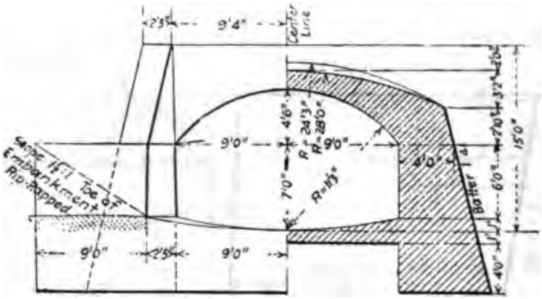
From Eng. Record, Vol. 50, p. 107



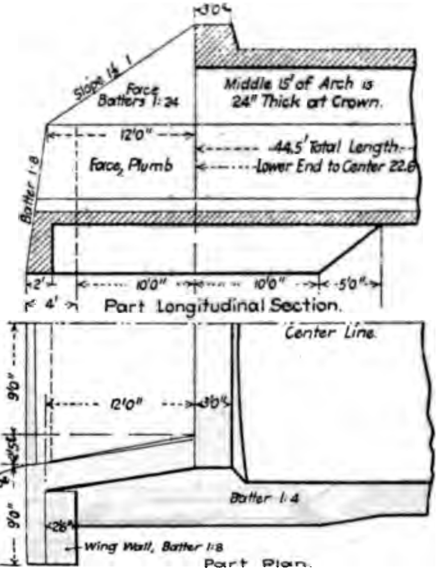


Type of Concrete Arch used on the St. Louis Kansas City & Colorado R. R.

From Eng. Record, Vol. 51, p 558



Half End Elevation. Half Transverse Section



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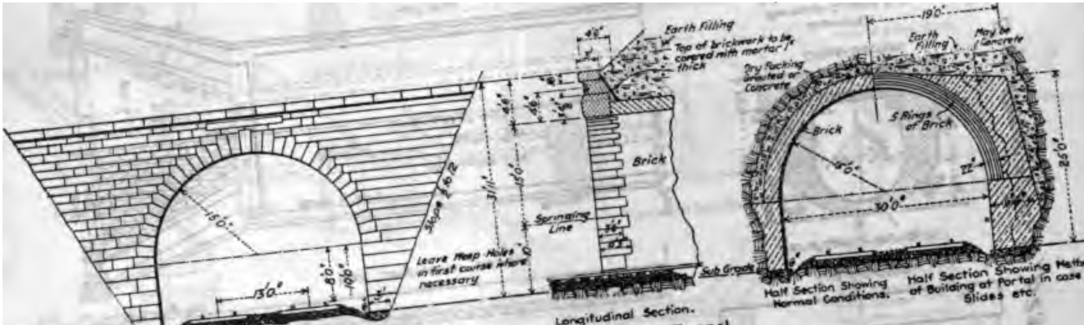
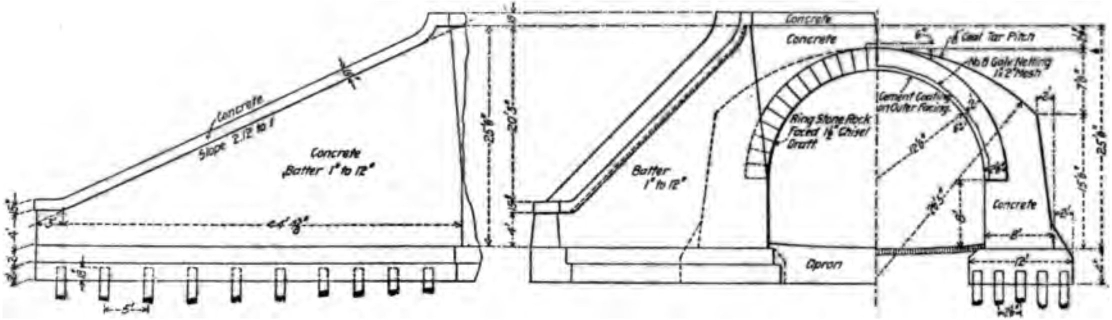
From Eng. Record, Vol. 50, p. 107



Standard B. & O. Double Track Tunnel.

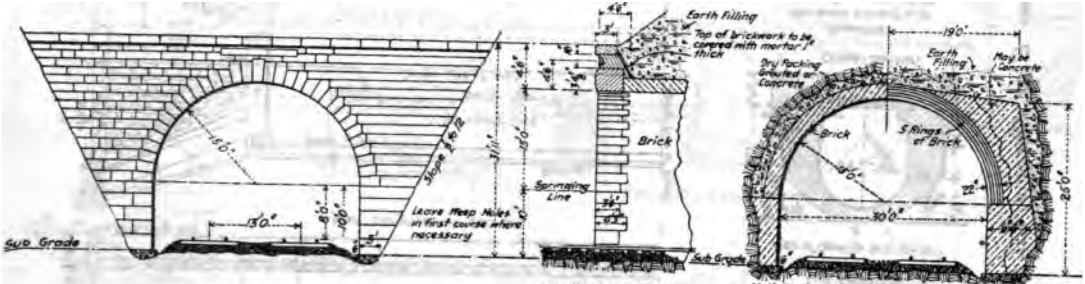
From R.R. Gaz., Vol. 38, p. 223



Elevation of Long Wing   Half End View   Half Section.

Details of Masonry; 25 ft. Arch Culvert at Dresden.

From R.R. Gaz., Mar. 14, 1902



Longitudinal Section.

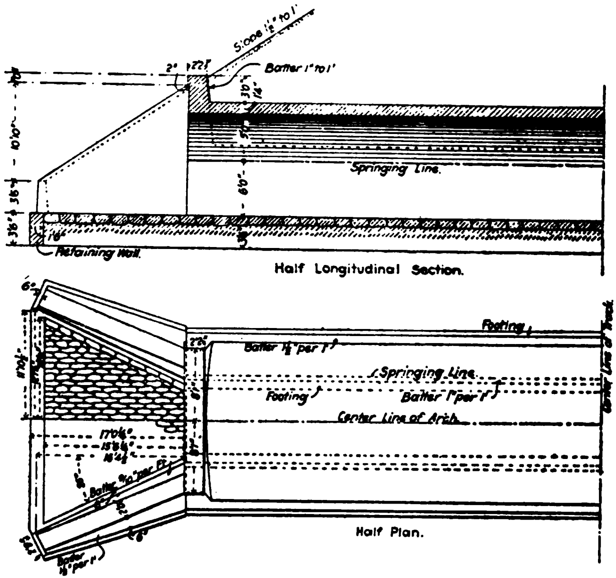
Half Section Showing Normal Conditions.

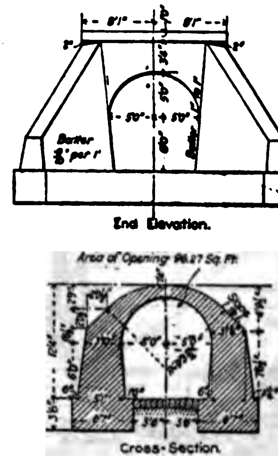
Half Section Showing Method of Building at Portal in case of Slides etc.

Standard B. & O. Double Track Tunnel.

From R.R. Gaz., Vol. 38, p. 223

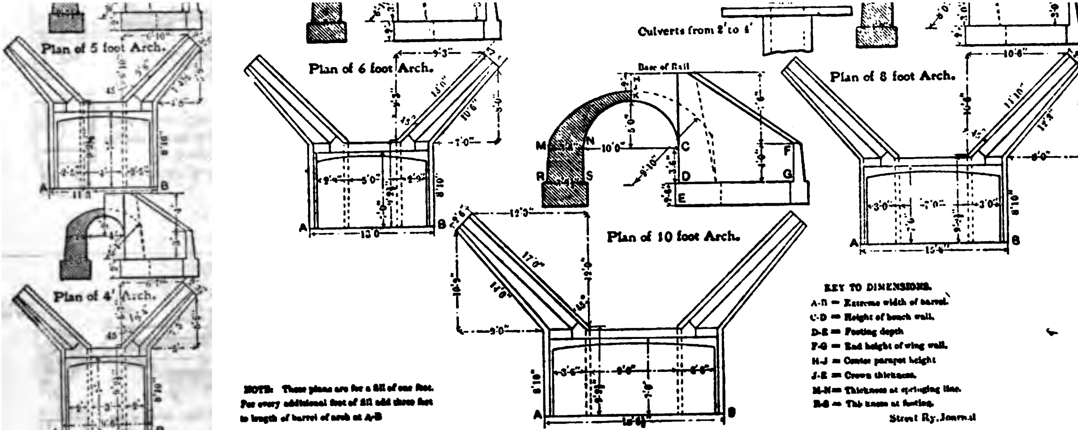
From Eng. Record, Vol. 51, p237





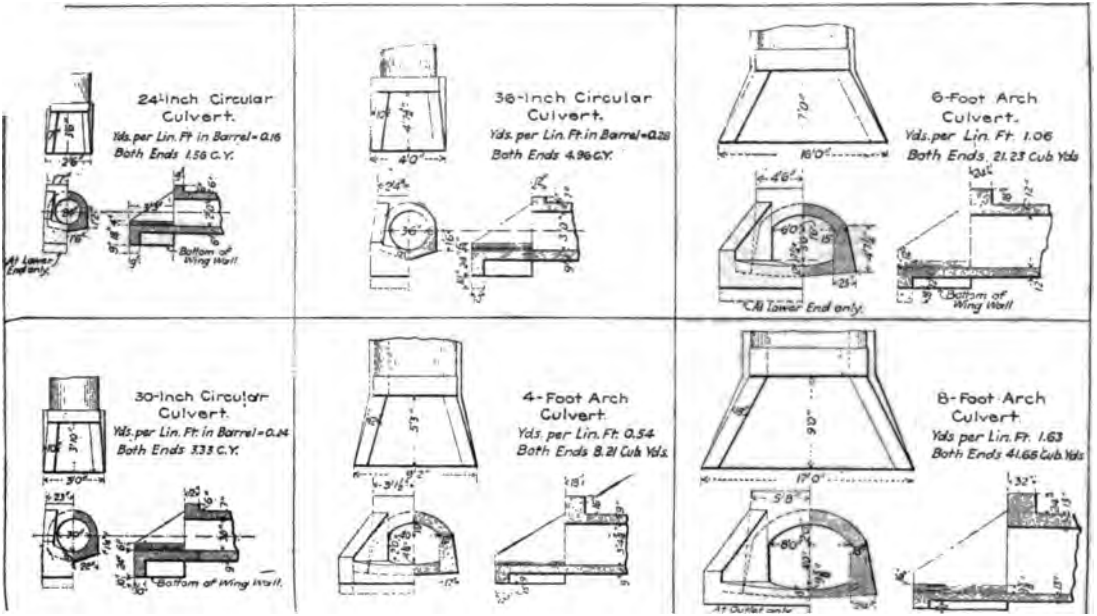
Quantities.	
For each Culvert	C. Y.
2 End Pavings	17.40
4 Wing Foundations	44.16
2 Retaining Walls	8.89
2 Parapet Walls	18.38
4 Wing Walls	70.44
Per Lineal Ft. of Arch	C. Y.
Paving	0.380
Foundations	1.707
[illegible]	[illegible]

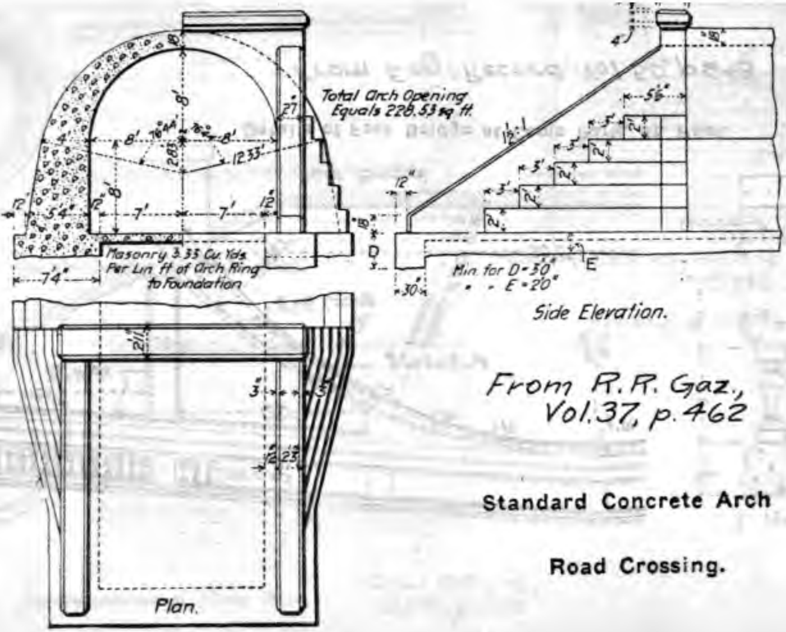
Standard Ten-Foot Concrete Arch, Western Maryland R. R.



PLANS OF STANDARD CONCRETE ARCH

Ft. Wayne & Wabash Valley Traction Co., From St. Ry. Journal, Vol. 28, p. 1035

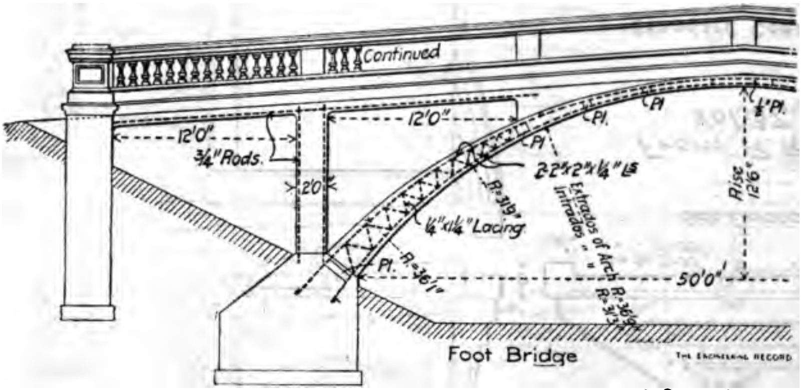

24-Inch Circular Culvert.
Yds. per Lin. Ft. in Barrel = 0.16
Both Ends 1.56 C.Y.
At Lower End only.
Bottom of Wing Wall
36-Inch Circular Culvert.
Yds. per Lin. Ft. in Barrel = 0.28
Both Ends 4.96 C.Y.
6-Foot Arch Culvert.
Yds. per Lin. Ft. 1.06
Both Ends 21.23 Cub. Yds.
C.W. Lower End only.
Bottom of Wing Wall
30-Inch Circular Culvert.
Yds. per Lin. Ft. in Barrel = 0.24
Both Ends 3.33 C.Y.
Bottom of Wing Wall.
4-Foot Arch Culvert.
Yds. per Lin. Ft. 0.54
Both Ends 8.21 Cub. Yds.
8-Foot Arch Culvert.
Yds. per Lin. Ft. 1.63
Both Ends 41.65 Cub. Yds.
At Outlet only

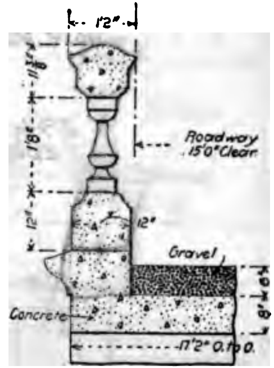


From R.R. Gaz.,
Vol. 37, p. 462

Standard Concrete Arch

Road Crossing.

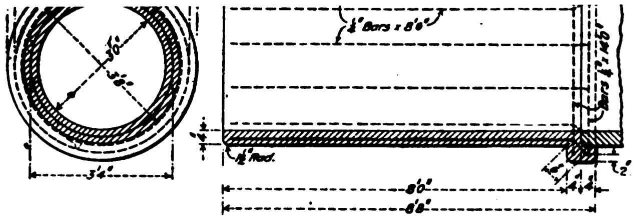




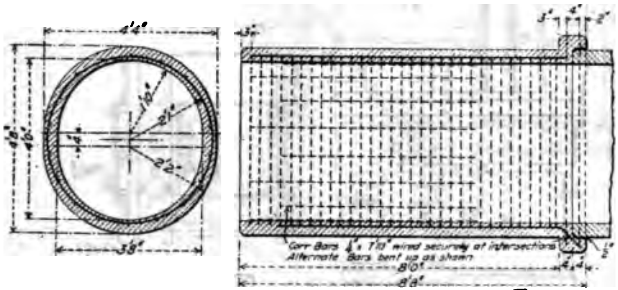
Balustrade.

Details of Foot Bridge at Como Park, St. Paul.

From Eng. Record, Vol. 50, p 648

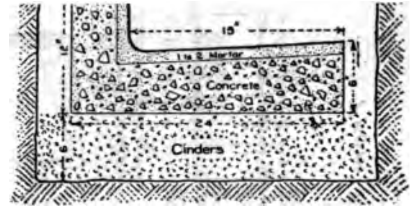


Reinforcement of 36-in. Pipe.



Reinforcement of 48-in. Pipe.

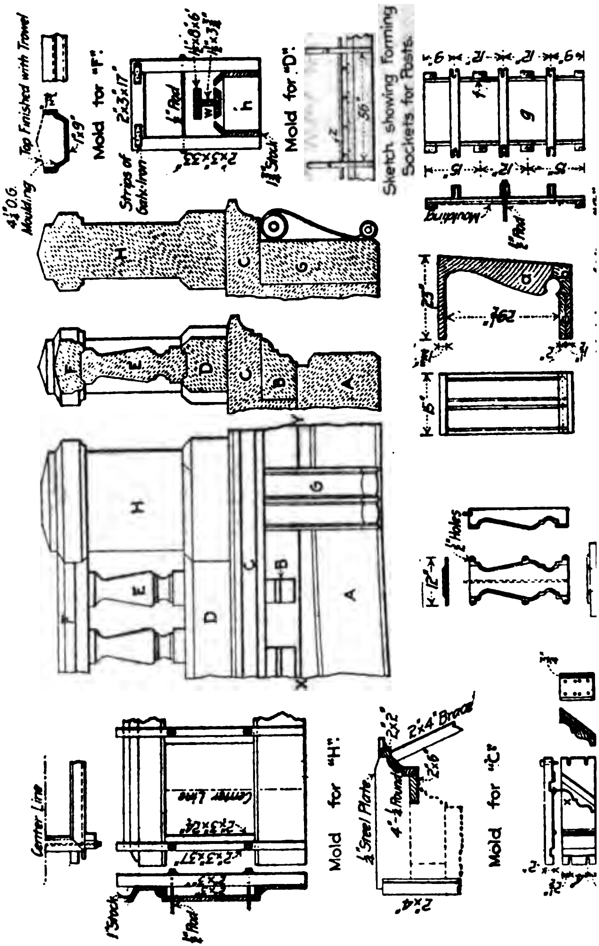
From R.R.Gaz., Vol. 41, p.309

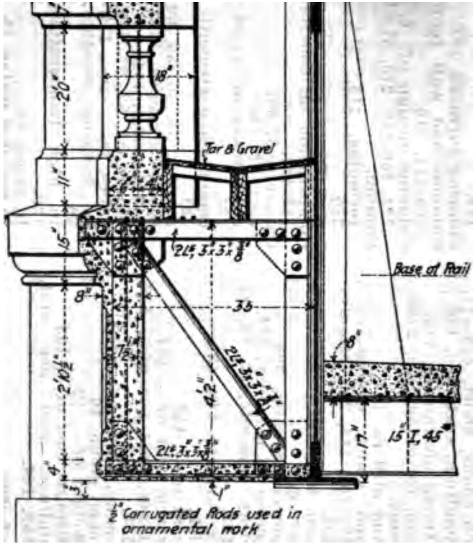


Concrete Curb and Gutter for Brick Pavement at Champaign, Ill.

From Eng. News, Vol. 49, p. 555

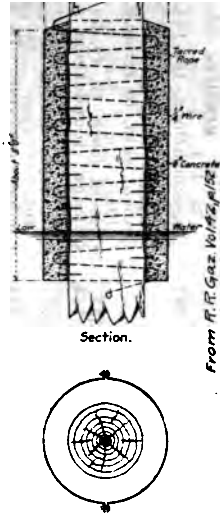
From Eng. News, Vol. 52, p. 491.





Section through Girder and Balustrade.

From R.R. Gaz., Vol. 37, p. 489



Section.

Plan.

Pile Protection by Concrete Sheathing, Durban, South Africa.

From R.R. Gaz. Vol. 42, p. 152

INDEX

AERIAL WIRE ROPE TRAMWAYS

MANUFACTURED BY

Leschen & Sons Rope Co., St. Louis, Mo.

ptation. Economical transportation of material such as Ore, Coal, Stone, Earth, Sand, Rock and Timber, from one place to another.

antages. Low cost of operation; small expense for maintenance; not affected by the elements; free from surface traffic; built in a straight line; requires no cuts, fills, bridges or winding detours.

th. Can be successfully operated over distances varying from a few hundred feet to practically any length beyond. The LESCHEN CO.'S PATENT AUTOMATIC Wire Rope Tramway built for the Penn-Wyoming Copper Co., Encampment, Wyoming, is 16 miles long.

acity. This may vary from a few tons to one hundred tons per hour, and in excess of this amount in many cases.

er uired or eloped. If inclination is sufficient and the loads descend, the tramway will operate by force due to gravity, the surplus power being controlled by brakes. Power, when necessary, is applied at either terminal station.

ems. Double Rope Tramway—Single Rope Tramway—Two-Bucket Tramway.

ble e nway. This is most economical type both in cost of operation and maintenance. Carriers travel on independent track ropes, are propelled and controlled by endless traction rope; towers support cable at intervals depending upon contour of ground, average spacing being 300 feet. LESCHEN CO.'S PATENT AUTOMATIC Tramway is an automatic type of the double rope system in which but one operator is required.

le nway. Endless wire rope to which carriers are positively attached; buckets loaded by Mechanical Loader; towers support cable at intervals; speed about 150 feet per minute. Not recommended for heavy capacity or long lengths.

-Bucket nway. Two carriers are used each traveling backward and forward on a track rope. A light endless traction rope used for propelling the carriers. This is simplest and least expensive system when line is short and contour of ground favorable.

tion oute. Tramway route should be surveyed in straight line connecting terminal points. Angles in horizontal plane to be avoided. If angle is absolutely necessary an extra station is required at angle point.

The A. LESCHEN & SONS ROPE COMPANY will furnish estimates of cost upon receipt of profile of line and definite data as to capacity per hour, class of material to be handled, weight of material per cubic foot, and terminal requirements.
An Approximate Estimate can be given, if profile is not procurable, upon receipt of the data mentioned.

ove data furnished by A. LESCHEN & SONS ROPE CO., St. Louis, Mo.

PATENT FLATTENED STRAND WIRE R

(A. Leschen & Sons Rope Co., St. Louis, Mo.)
SOLE MANUFACTURERS


A—(5 x 28)


B—(6 x 25)


C—(5 x 9)


D

HOISTING ROPES HAULAGE ROF

With the view of increasing the wearing surface of wi thereby prolonging the life of the rope, A. Leschen & Sons Ro had on the market for many years a patent flattened strand w which they are sole makers. The illustrations given above sh sections the construction of this style of rope. These flatt wire ropes are made in various grades of material. The satisfa wire ropes have given justifies the manufacturers' claim for over any wire rope manufactured. The accompanying illustra the comparative wearing surface between patent flattened str round or ordinary construction of wire rope.


PATENT FLATTENED STRAND


ROUND STRA

We give herewith Table of Weights, Breaking Strains, I etc., commencing with the highest grade:

"HERCULES" PATENT FLATTENED STRAND.

HOISTING				HAULAGE		
Diameter in Inches	*Price per foot in cents	Approximate breaking strain in tons of 2,000 lbs.	Average weight per ft.	Diameter in Inches	Price per foot in cents	Approximate breaking strain in tons
½	20¾	13½	.44	½	16¼	13
⅝	28	22½	.73	⅝	25	21
¾	37½	32	1.00	¾	35	30
⅞	49	40½	1.35	⅞	44	38
1	60	56	1.80	1	58	53
1⅛	71	67	2.30	1⅛	70	64
1¼	89	84	2.80	1¼	88	80
1½	137	124	4.00			
1¾	208	168	5.40			
2	225	211	7.50			
2¼	285	260	9.25			

* The purpose of giving these list prices is to give an idea of c *value.*

A. LESCHEN & SONS ROPE CO.

SOLE MANUFACTURERS

PATENT FLATTENED STRAND WIRE ROPES

CRUCIBLE CAST STEEL HOISTING ROPE

Inches	Price per Foot in cents	Approximate Breaking Strain in Tons of 2,000 lbs.	Average Weight per Foot	Minimum size of Drums or Sheaves in ft.
½	14½	9½	.44	1½
⅝	18¼	15	.73	2
¾	24	21	1.00	3
⅞	30	29	1.35	3½
	39½	38	1.80	4
⅛	50	47	2.30	4½
¼	59½	56	2.80	5
⅜	73	69	3.40	5½
½	86	81	4.00	5¾
⅝	96	94	4.75	6¼
¾	121	109	5.40	7¼
	144	140	7.50	8
¼	182	176	9.25	9

SPECIAL STEEL HOISTING ROPE

Diameter in Inches	Price per Foot in cents	Approximate Breaking Strain in Tons of 2,000 lbs.	Average Weight per Foot	Minimum size of Drums or Sheaves in ft.
½	17½	11 1/16	.44	1½
9/16	19½	14	.54	1¾
⅝	22½	17	.73	2
¾	30	24½	1.00	3
⅞	38	33	1.35	3½
1	48	43	1.80	4
1⅛	59	54	2.30	4½
1¼	70	64	2.80	5
1½	105	93	4.00	5¾
1¾	155	127	5.40	7¼
2	177	162	7.50	8
2¼	220	204	9.25	9

HAULAGE ROPE (Crucible Cast Steel)

Inches	Price per Foot in cents	Approximate Breaking Strain in Tons of 2,000 lbs.	Average Weight per Foot	Minimum size of Drums or Sheaves in ft.
⅜	7	5	.25	
½	10	9	.44	
⅝	14	14	.73	
¾	20½	20	1.00	
⅞	27½	27	1.35	
	35	36	1.80	
⅛	45	45	2.30	
¼	54	54	2.80	

HAULAGE ROPE (Special Steel)

Diameter in Inches	Price per Foot in cents	Approximate Breaking Strain in Tons of 2,000 lbs.	Average Weight per Foot	Minimum size of Drums or Sheaves in ft.
⅜	11	6 1/16	.25	2
½	14	10½	.44	2¼
⅝	18	16½	.73	3¼
¾	27	23	1.00	4½
⅞	35	31	1.35	5
1	45	40	1.80	5¾
1⅛	54	50	2.30	6¼
1¼	68	62	2.80	7¼

These ropes are made of the best rucible steel wire, combining in a igh degree ductility and tensile rength.

As the name indicates, this rope is made from a special grade of steel, combining high tensile strength with flexibility, without a tendency to brittleness.

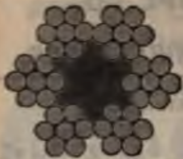
PATENT FLATTENED STRAND SWEDES IRON

HOISTING

Inches	Price per Foot in cents	Approximate Breaking Strain in Tons of 2,000 lbs.	Average Weight per Foot	Minimum size of Drums or Sheaves in ft.
½	10½	4	.38	2
⅝	15½	6	.57	3
¾	21	9	.83	3½
⅞	26	13	1.20	4
1	34	17	1.58	4¾
1⅛	43	21	2.00	5¼
1¼	52	28	2.50	5¾
1⅜	62½	34	3.00	6¼
1½	74	40	3.65	6¾
1⅝	82	45	4.15	7½
1¾	104	54	5.00	9
2	120	66	6.30	10¾
2¼	152	75	8.00	11½

HAULAGE

Diameter in Inches	Price per Foot in cents	Approximate Breaking Strain in Tons of 2,000 lbs.	Average Weight per Foot	Minimum size of Drums or Sheaves in ft.
½	8¼	4½	.38	3½
⅝	12½	7	.60	4¾
¾	17½	10	.87	6
⅞	22	13½	1.20	6¾
1	29	18	1.58	7¾
1⅛	36½	22½	2.00	8½
1¼	45	27	2.50	9½



GALVANIZED IRON WIRE ROPE

7 WIRES — FOR SHIPS' RIGGING, GUYS FOR DERRICKS, ETC. — 12 WIRES

Approximate diameter in inches	Circumference in inches	Estimated weight per foot Hemp center	Price per foot, in cents	Circumference in inches of new Manila Rope of equal strength	Breaking strain tons of 2,000 lbs.	Approximate diameter in inches	Circumference in inches	Estimated weight per foot Hemp center	Price per foot, in cents	Circumference in inches of new Manila Rope of equal strength
6 STRANDS, 7 WIRES										
2	6	6.00		12	50	1	3	1.44	14	5¾
1¾	5½	4.85	44	11	44	⅞	2¾	1.21	12	5¼
1 11/16	5¼	4.40	41	10½	40	13/16	2½	1.00	10	5
1⅝	5	4.00	38	10	36	¾	2¼	0.81	9	4¾
1½	4¾	3.60	35	9½	32	⅝	2	0.64	8	4½
1 7/16	4½	3.25	31	9	29	9/16	1¾	0.49	7	3¾
1⅜	4¼	2.90	27	8½	26	½	1½	0.36	6	3
1¼	4	2.55	24	8	23	7/16	1¼	0.25	5	2½
1 3/16	3¾	2.25	21	7½	20	⅜	1⅛	0.20	4	2¼
1⅛	3½	1.95	18	6½	18	5/16	1	0.16	3½	2
1 1/16	3¼	1.70	16	6	15					
5 STRANDS, 7 WIRES										
9/32	⅞	0.123	3	1¾	1.1	7/32	⅝	0.063	2¼	1¼
¼	¾	0.090	2½	1½	0.81	3/16	½	0.040	2	1⅛
6 STRANDS, 12 WIRES										
2	6	6.00		12	50	1⅜	4¼	2.90	29	8½
1¾	5½	4.85	46	11	44	1¼	4	2.55	25	8
1 11/16	5¼	4.40	43	10½	40	1 3/16	3¾	2.25	22	7½
1⅝	5	4.00	40	10	36	1⅛	3½	1.95	19	6½
1½	4¾	3.60	37	9½	32	1 1/16	3¼	1.70	17	6
1 7/16	4½	3.25	33	9	29	1	3	1.44	15	5¾

'ABLE SHOWING BREAKING STRAINS, WEIGHTS, 'RICES, ETC., OF ROUND STRAND WIRE ROPES

FOR

HOISTING AND HAULAGE

MANUFACTURED BY

A. LESCHEN & SONS ROPE CO., ST. LOUIS, MO.

HOISTING ROPE—6 Strands, 19 Wires

Average weight per foot	Breaking Strain in Tons of 2,000 lbs.					Minimum size of Drum in Feet					List Prices per Foot in Cents					Diameter in Inches
	Hercules ‡	Special Steel	Crucible	Plough	Swedes Iron	Hercules	Special Steel	Crucible	Plough	Swedes Iron	Hercules	Special Steel	Crucible	Plough	Swedes Iron	
9.85	266	222	190	254	95	13½	9½	9½	13½	15	262	210	175	250	140	2½
8.	238	182	156	208	78	12	8½	8½	12	13	229	170	142	200	117	2¼
6.30	191	144	124	165	62	11	8	8	11	12	181	134	111	156	92	2
4.85	157	112	96	128	48	9	7¼	7¼	9	10	166	115	93	135	80	1¾
4.15	128	97	84	111	42	8½	6¼	6¼	8½	8½	125	91	74	108	63	1⅝
3.55	113	84	72	96	36	8	5¾	5¾	8	7½	109	80	66	93	57	1½
3.	96	72	62	82	31	7½	5½	5½	7½	7	90	67	56	77	48	1⅜
2.45	76	58	50	67	25	7	5	5	7	6½	71½	55	46	68	40	1¼
2.	60	49	42	56	21	6	4½	4½	6	6	57½	45	38	52	33	1⅛
1.58	50	39	34	44	17	5	4	4	5	5¼	49	36	30	43	26	1
1.20	36	30	26	34	13	4½	3½	3½	4½	4½	39	28	23	34	20	⅞
.89	29	22	19 4/10	25	9 7/10	4	3	3	4	4	30	22	18	26	16	¾
.62	20	15 8/10	13 8/10	18	6 8/10	3½	2¼	2¼	3½	3½	22½	16½	14	19	12	⅝
.50	17	12 7/10	11	14½	5½	3	1¾	1¾	3	2¾	20	14	12	16	10	9/16
.39	12½	10 1/10	8 8/10	11 4/10	4 4/10	2¾	1½	1½	2¾	2¼	16½	12½	11	14	08	½
.30	10	7 8/10	6 8/10	8.85	3 4/10	2½	1¼	1¼	2½	2	15	11½	10	13	07½	7/16
.22	7	5 78/100	5	6.55	2 8/10	2	1	1	2	1½	14½	11	09½	12½	07	⅜
.15	......	4 5/100	3 4/10	4½	1 7/10	1½	⅔	⅔	1½	1	....	10¾	09¼	12¼	06¾	5/16
.10	......	2 7/10	2 4/10	3	1 2/10	1	½	½	1	¾	....	10½	09	12	06½	½

‡ "HERCULES" made from a specially drawn and patented steel, ɪich is made exclusively for A. Leschen & Sons Rope Co., Sole Makers.

TABLE—CONTINUED

SHOWING BREAKING STRAINS, WEIGHTS, PRI(
ETC., OF ROUND STRAND WIRE ROPES

FOR

HOISTING AND HAULAGE

MANUFACTURED BY

A. LESCHEN & SONS ROPE CO., ST. LOUIS, M(

HAULAGE ROPE—6 Strands, 7 Wires

Diameter in Inches	Average weight per foot	Breaking Strain in Tons of 2,000 lbs.					Minimum size of Drum in Feet					List Prices pe Foot in Cent			
		Hercules ‡	Special Steel	Crucible	Plough	Swedes Iron	Hercules	Special Steel	Crucible	Plough	Swedes Iron	Hercules	Special Steel	Crucible	Plough
1½	3.55	......	79	68.	91.	34.	10	8½	8½	10	13	......	75	60	90
1⅜	3.	......	68	58.	78.	29.	9¾	8	8	9¾	12	......	64	51	75
1¼	2.45	74	56	48.	64.	24.	9¼	7¼	7¼	9¼	10¾	70½	53	43	61
1⅛	2.	58	46	40.	53.	20.	8	6¼	6¼	8	9½	56	44	36	51
1	1.58	47	37	32.	42.	16.	6¼	5¾	5¾	6¾	8½	46	34	28	41
⅞	1.20	33½	28	24.	32.	12.	6	5	5	6	7½	35	26	22	32
¾	.89	25½	21	18 6	24.	9.3	5¼	4½	4½	5¼	6¾	28	20	16	25
11/16	.75	......	18.4	15.8	21.	7.9	5	4	4	5	6	......	17	13½	20
⅝	.62	18½	15.1	13.2	17.	6.6	4½	3½	3½	4½	5¼	20	14	11	17
9/16	.50	......	12.3	10.6	14.	5.3	4	3	3	4	4½	......	11½	9	14
½	.39	11½	9.70	8.4	11.	4.2	3¾	2½	2½	3¾	4	13	9½	7½	11
7/16	.30	......	7.50	6.6	8.55	3.3	3½	2¼	2¼	3½	3¼	......	7¼	6½	8
⅜	.22	......	5.58	4.8	6.35	2.4	3¼	2	2	3¼	2¾	......	6	5½	6½
5/16	.15	......	3.88	3.4	4.35	1.7	3	1¾	1¾	3	2½	......	5½	4½	6
9/32	.125	......	3.22	2.8	3.65	1.4	2¾	1½	1½	2¾	2¼	......	5	4	5½

‡ "HERCULES" made from a specially drawn and patented which is made exclusively for A. Leschen & Sons Rope Co., Sole M